1983 Ontario Highway Bridge Design Code

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Ontario
Highway
Bridge
Design
Code

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Documentation

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Preface to Second Edition

The First Edition of the Ontario Highway Bridge Design Code was issued in 1979, with P.F. Csagoly as Chairman of the Code Development Committee. It was published by the Ontario Ministry of Transportation and Communications for use in the Province of Ontario, and was amended by the subsequent issuing of three Addenda, in September 1979, December 1980 and May 1982. The Code was written in SI units and limit states format and, because of the many changes in approach compared to codes previously used in Ontario, it was issued on a trial basis. It has been applied to the design and evaluation of virtually all bridges on the King's Highways system since 1979, and has also been used in most of the larger municipal jurisdictions.

The Second Edition represents an extensive rewrite of the Code and Commentary based on four years experience of application of the First Edition and new research and development information. The number of sections has been reduced from 17 to 14 by combining previous Sections 2, 3, 4 and 17 into the new Section 2. The following changes in section numbers have also been made: Section 5 becomes Section 3, Section 15 becomes Section 4, and Section 16 becomes Section 5. While modifications have been made to all sections, the most significant changes appear in Sections 2, 3, 6 and 13.

Compliance with this Second Edition of the Code will become a requirement for the design and evaluation of all bridges on the public road system in Ontario at a date to be announced by the Minister. The Commentary will remain an advisory document.

General Information

The Code is contained in this volume, and consists of the Overall Requirements, Sections 1 to 14, and the Appendices. The Commentary is contained in an advisory volume which explains the background to the Code provisions and lists the references used in writing the Code. The publications referenced in the Commentary are filed in the library of the Ministry of Transportation and Communications, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8.

The Code is divided into the following parts:

Overall Requirements
Section 1 — General Features
Section 2 — Loads
Section 3 — Analysis of Bridge Superstructures
Section 4 — Deck Joints and Bearings
Section 5 — Accessories
Section 6 — Substructures and Retaining Walls
Section 7 — Concrete Deck Slabs
Section 8 — Reinforced Concrete
Section 9 — Prestressed Concrete
Section 10 — Structural Steel
Section 11 — Structural Fatigue and Fracture Control
Section 12 — Soil-Steel Structures
Section 13 — Wood Structures
Section 14 — Evaluation of Existing Bridges

The Code is based upon the limit states design philosophy, and does not recognize the provisions of the working stress method alone as appropriate for the design or evaluation of highway bridges. It is expected that the adoption of this design philosophy will result in structures with a more uniform level of safety and in an effective reduction in costs. The advantages of limit states design are reinforced in this Code by a rational approach to live loads, representative of actual highway traffic and based upon extensive load surveys.

The basic requirement at the ultimate limit state is that the factored resistance shall exceed the total factored load effect. The load factors applied to the loads and the performance factors applied to the resistances were derived using a calibration process based on a service life of 50 years for new bridges. All loads and load factors are given in Section 2 for serviceability and ultimate limit states. The performance factors are material-related and are, thus, given in the appropriate sections for the various ultimate limit state conditions. It should be noted that performance factors are not applied for the serviceability limit states.

Section 6 has been prepared with the expectation that there will be a continuing dialogue between the engineer and the geotechnical specialist for all aspects of foundation work, covering both the design and the construction phases.

The Code is not intended to be a restrictive design aid, but to be a document to prescribe the limits and basic requirements of design. The engineer is expected to utilize new developments in structural engineering whenever they appear desirable and advantageous, and provision is made for the approval of methods not prescribed in the Code.

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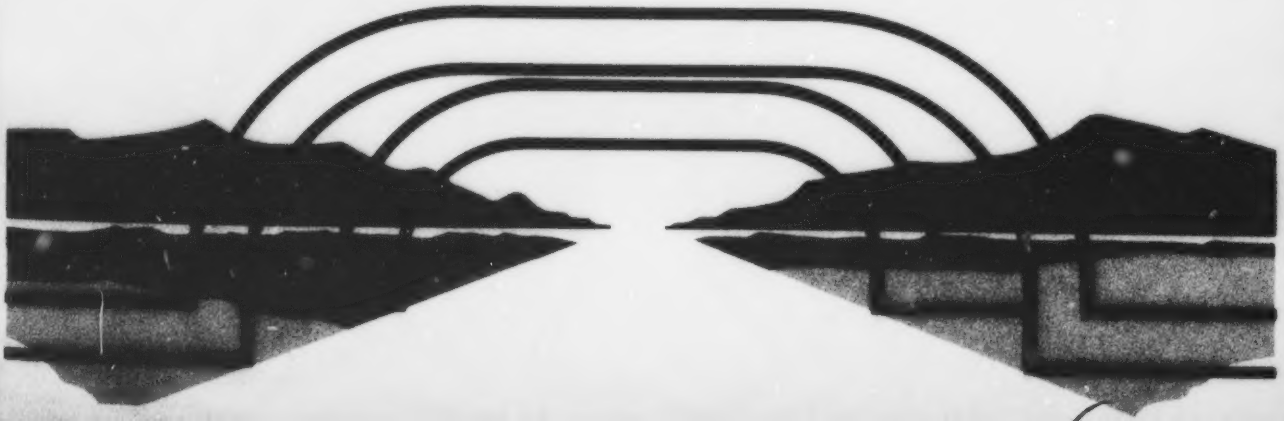

0 Overall Requirements

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Scope of Code

0—1

This Code governs the design, evaluation and structural rehabilitation of highway bridges in the Province of Ontario with individual spans less than 125 m in length. In addition to highway bridge components and accessories, it covers pedestrian bridges, retaining walls, barriers, poles and sign supports, but does not fully cover cable-supported structures, movable bridges or masonry bridges.

The Code does not govern public utility structures, interprovincial bridges, international bridges, or bridges used solely for railway or rail transit purposes.

General Definitions

0—2

These definitions are used throughout the Code. Specific definitions are given in each section.

Approval — approval in writing by the Minister, unless otherwise stated.

Approved — approved in writing by the Minister, unless otherwise stated.

Box Girder Bridge — a bridge whose transverse cross-section is of rectangular or trapezoidal cellular form, with the individual cells having interior dimensions of at least 0.8 m both vertically and horizontally.

Bridge — any structure having a span of 3 m or more that forms part of a highway or over, or under which the highway passes.

Checker — a member or licensee of the Association of Professional Engineers of Ontario, who carries out the design check of a bridge or structure.

Collapse — a major change in the geometry of the structure, rendering it unfit for use.

Component — a structural element or combination of elements requiring individual design consideration.

Constructor — a person who contracts to undertake all of the work on a project; or an owner who contracts with more than one person for parts of the work on a project, or who undertakes all or part of the work on a project himself.

Design — includes planning, computing, dimensioning, drawing, and writing specifications for the construction of new bridges or structures and for the evaluation or rehabilitation of existing bridges.

Engineer — a member or licensee of the Association of Professional Engineers of Ontario, who carries out the design of a bridge or structure.

Evaluation — the determination of the load-carrying capacity of a bridge.

Factored Resistance — the product of the unfactored resistance and the applicable performance factor.

Failure — a state in which rupture, severe distortion or loss of strength has occurred as a result of the load-carrying capacity of a component or connection having been exceeded.

Highway — a common and public highway and including a street, avenue, parkway, driveway, square, place, bridge, viaduct or trestle, designed and intended for, or used by, the general public for passage of pedestrians, animals, or vehicles.

Highway Classification — highway classes as given in Table 2—4.3.1.1.

Limit States — those conditions beyond which the structure or component ceases to fulfil the function for which it was designed.

Load Effect — any effect or response of a structural component due to loads, forces, imposed deformations and volumetric changes.

Load Factor — a factor applied to loads to take into account variability of loads, lack of precision in analysis for load effects, and reduced probability of loads from different sources acting simultaneously.

Minister — the Minister of Transportation and Communications, Ontario, or his nominee.

Ministry — the Ministry of Transportation and Communications, Ontario.

Multi-Load Path Structure — a structure in which there is no component or connection the failure of which would cause the collapse of the structure under the action of its own weight and that of normal traffic.

Owner — a person having jurisdiction and control over the bridge.

Performance Factor — a factor applied to the unfactored resistance at the ultimate limit state to take into account variability of material properties and dimensions, workmanship, type of failure, and uncertainty in the prediction of resistance.

Person — an individual, board, commission, partnership or corporation, including a municipal corporation, and employees, agents, successors, and assigns of any of them.

Plans — all drawings, descriptions, and specifications being parts of a contract and all drawings and descriptions produced by the constructor for the erection of a bridge or structure, and all revisions thereto.

Rehabilitation — repairs affecting the serviceability, durability, and strength of the structure.

Safety Index — a numerical assessment of margin of safety expressed as a number of standard deviations from a mean.

Serviceability Limit States — those limit states which restrict the intended use of a structure, including fatigue, vibration, permanent deformation, and cracking.

Span — the minimum horizontal distance between adjacent supports of the superstructure of a bridge, or the longest horizontal dimension of the cross-section of a culvert taken perpendicular to the walls, unless defined otherwise.

Substructure — the abutments, wingwalls, retaining walls, piers, columns, bearings, and foundations of a bridge.

Superstructure — all parts of a bridge above the substructure.

Ultimate Limit States — those limit states which concern structural safety, including failure, overturning and instability but excluding fatigue.

Unfactored Resistance — the resistance of a component or connection, based upon specified material properties and nominal dimensions, and calculated in accordance with the Code.

General Provisions 0—3

0—3.1 The plans shall indicate all necessary details of the bridge, the strength of materials, the classification of the highway, and the date of the relevant edition of the Code.

0—3.2 In order to validate and certify the design, the engineer and the checker shall each affix his Association of Professional Engineers of Ontario seal to the plans, and sign them.

0—3.3 Unless otherwise stated in the Code, construction shall be in accordance with "General Conditions, Standard Specifications and Supplemental Specifications" of the Ministry, or other approved specifications.

0—3.4 The constructor shall provide the owner with adequate plans sealed and signed by a professional engineer indicating temporary supports and the method and sequence of construction.

0—3.5 When the method of construction, as envisaged by the engineer, is of such nature as to cause critical load effects during construction, a sequence of construction shall be indicated on the plans.

0—3.6 The engineer shall obtain approval for the introduction of any item, component, process, analytical method, or method of construction that is not covered by or is in conflict with the Code.

0—3.7 Structures shall be designed having regard to overall economy, and the needs of fabrication, construction, erection, and maintenance.

0—3.8 Structures shall be designed having regard to ease of inspection, durability and protection against environmental effects.

0—3.9 Consideration shall be given to the environmental effects produced by the structure on its surroundings, and care shall be exercised to keep disadvantageous effects to a practical minimum.

0—3.10 In interpreting the clauses of the Code, the masculine shall be interpreted to include the feminine, and the singular to include the plural, as appropriate. Cross-reference to a clause shall be interpreted to mean reference to that clause in its entirety.

Limit States Methods

0—4

0—4.1 Structural components and connections shall be proportioned to satisfy both the ultimate limit states and serviceability limit states requirements. The engineer shall ensure that all limit states appropriate to the particular structure being designed are investigated.

0—4.2 Elastic methods of analysis are generally prescribed for determining the response of a structure at all limit states, and inelastic methods require approval unless specifically allowed by the Code.

Design of New Bridges

0—5 All sections of the Code except Section 14 shall apply to the design of new bridges.

Evaluation of Existing Bridges

0—6 All sections of the Code may be applied to the evaluation of the load-carrying capacity of existing bridges. Section 14 prescribes specific clauses and modifications for evaluation, to be used in conjunction with the other sections. Consideration shall be given to the anticipated future service life of the bridge when assessing evaluation results.

Rehabilitation Design of Existing Bridges

0—7 All sections of the Code may be applied to the rehabilitation design of existing bridges. If a bridge is to be rehabilitated for a further 50-year service life, the Code provisions for the design of a new bridge may be applied. If a bridge is to be rehabilitated for a further service life of significantly less than 50 years, or for a prescribed load or posting level, the Code provisions for the evaluation of existing bridges, including Section 14, may be applied. The design criteria used for a bridge rehabilitation project shall be submitted for approval.

Construction

0—8 The appropriate sections of the Code shall apply to the calculations required at various stages of construction. The Code shall also apply to temporary structures. Section 2 provides, in general, for construction loads and loads on temporary structures but additional criteria may need to be developed and approved for unusual construction techniques or rehabilitation projects.

Application of Code

0—9

0—9.1 The limits of application of the Code are given in Clause 0.1 of this section. For bridges outside these limits, the provisions of the Code shall apply when appropriate, and additional design criteria shall be submitted for approval.

0—9.2 For movable bridges, the Code shall apply to all functions except opening and closing.

0—9.3 The provisions of the Code concerning railings shall apply unless otherwise specified by the owner.

0—9.4 Model testing for design, and bridge load testing for evaluation shall be carried out under the supervision of an engineer having extensive experience in these types of activity.

0—9.5 A structure other than a multi-load path structure shall not be used unless approved.

1 General Features

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Reference Publications

Section 1 makes reference to the following publications:

Roads and Transportation Association of Canada

- *Guide to Bridge Hydraulics*, 1973, as revised by Metric Revision Supplement, 1980.

Ministry of Transportation and Communications

- *MTC Design Flood Criteria*, MTC Provincial Highways Directive B-100, 80 10 16.
- *MTC Drainage Manual*, Chapter B, "Design Flood Estimation for Small Watersheds," 80 11.
- *MTC Drainage Manual*, Chapter D, "Hydraulic Design of Culverts," 82 03.
- *MTC Bridge Hydraulics Manual*, Chapter C, "Design Flood Estimation for Medium and Large Watersheds," 79 12.

Scope 1 — 1

This section provides information for establishing the basic geometry of structures, which are thereafter proportioned by application of other sections of the Code. Dimensional requirements for bridge deck widths, bridge openings and bridge railings are given, together with requirements pertaining to deck drainage, roadway surfaces, approach slabs and utilities. Minimum geometric requirements to facilitate inspection and maintenance operations are also covered.

Hydraulic design requirements for bridges and culverts and their appurtenant works are specified, together with provisions for the prevention of damage to structures and adjacent property by flooding and erosion.

Definitions 1 — 2

Abnormal Flood Stage — the flood stage produced by factors other than the channel slope and the boundary roughness of the channel and flood plain.

Abutment — a structural component supporting the end of a bridge superstructure and retaining the roadway fill.

Apron — protective material laid on a streambed to prevent scour at a bridge pier, abutment, culvert outlet, toe of a slope or similar location.

Armour Coat — a natural layer of stone protecting erodible material underlying the bed of a channel.

Arterial Road — a road primarily for through traffic.

Auxiliary Lane — the portion of a roadway adjoining the through lanes for speed change lanes, truck climbing lanes or other purposes supplementary to through traffic movement.

Backwater — the rise of water level upstream due to an obstruction or constriction in a channel.

Ballast Wall — the portion of the abutment between the end of the superstructure and the approach fill.

Barrier Wall — a wall, with or without rails, for the purpose of confining vehicles to the roadway.

Bikeway — any road, path or way which is specifically designated as being open to bicycle travel, whether exclusive or shared with other modes of transportation.

Box Culvert — a type of closed invert culvert having a rectangular cross-section and constructed of concrete or timber.

Clearance — the unobstructed distance between two objects.

Closed Invert Culvert — a culvert having an invert which is structurally integral with the walls.

Collar — a concrete ring surrounding the end of a metal culvert.

Collector Road — a road on which traffic movement and access to property have similar importance.

Combination Railing — a railing on a bridge or retaining wall to contain both vehicular and pedestrian traffic.

Competent Velocity — the velocity of water which can just move a specified type or size of material on a streambed.

Crossfall — the transverse slope of a roadway surface.

Crown — the highest point of a roadway cross-section, or the highest point of the interior of an arch or pipe at a given cross-section.

Culvert — for the purpose of this section, a conduit, usually covered by fill, whose primary function is to convey surface water through an embankment.

Culvert Uplift — the upward movement of a culvert end due to buoyancy forces.

Curb — a raised edge along the roadway on a bridge deck.

Cutoff — a buried vertical wall below the end of a culvert or other structure.

Deck Joint — a structural discontinuity between two elements, at least one of which is a deck element, designed to permit relative rotation and/or translation between the two.

Deck Length — the distance between abutment deck joints, or between ends of the bridge deck if there are no joints, measured along the centreline of highway.

Degradation — the progressive general lowering of a stream channel over a long period by erosion other than that caused by a constriction.

Depth of Scour — the depth of material removed from a streambed by scour, measured below the average original bed elevation.

Design Discharge — the discharge which a structure is designed to accommodate without exceeding the adopted design constraints.

Discharge — the rate of flow of water in cubic metres per second.

Downpipe — a pipe to convey water from a deck drain to the ground or to a storm sewer.

Downspout — a short pipe extending from a deck drain below the bridge superstructure.

Drop Structure — a structure in a culvert or channel which permits water to drop to a lower level.

Flood — a relatively high flow in terms of either water level or discharge.

Flood Plain — the relatively level land which adjoins a watercourse or other body of water and which is subject to periodic flooding unless protected artificially by a dike or similar structure.

Freeboard — the height from a design water level to the top of an embankment, roadway, dam or wall.

Freeway — a fully controlled-access road exclusively for through traffic.

Gabion — a wire mesh basket filled with stones or broken concrete and forming part of a larger unit used for erosion control or other purposes.

General Scour — scour which occurs in a waterway opening as a result of constriction of the flow.

Guiderail — a barrier along the edge of a roadway to assist in restraining vehicles from leaving the roadway.

Haunch — for the purpose of this section, the thickened portion of a bridge deck immediately over a bridge girder.

Headwall — a wall at the end of a culvert normally extending from the invert to above the soffit or crown of the culvert, parallel to the roadway or normal to the longitudinal axis of the culvert.

Headwater Elevation — the water level upstream from a structure.

Improved Inlet — a culvert inlet incorporating inlet geometry refinements other than those used in conventional culvert practice, for the purpose of improving the culvert capacity.

Invert — the streambed or floor within a structure or channel.

Local Road — a road primarily for access to property.

Local Scour — scour which occurs at a pier or abutment as a result of local obstruction to the flow, and measured below the level of general or natural scour adjacent to the pier or abutment.

Median — the portion of a divided highway separating roadways carrying traffic in opposite directions.

Median Size — the size of bed material which exceeds and is exceeded by 50% of the material by weight.

Natural Scour — the scour of a streambed resulting from natural phenomena such as channel meandering.

Normal Design Flood — the design flood used for the hydraulic design of structures in the absence of imposed criteria such as the regulatory flood.

Normal Water Level — the average summer water level.

Open Deck Joint — a deck joint designed to permit the free flow of water through the joint.

Open Footing Culvert — a culvert having either a natural invert or an artificial floor not integral with the walls.

Original Streambed — the bed before artificial constriction of the channel.

Partial Headwall — a headwall extending from the invert to less than the full height of a culvert.

Pier — a substructure, other than an abutment, which supports a bridge superstructure.

Piping — the subsurface erosion caused by movement of water through fill or natural ground.

Railing — a barrier consisting of rails and supports; a barrier wall.

Ramp Bridge — a bridge on a ramp to or from a highway.

Regional Flood — a flood generated by a storm designated by the Ontario Ministry of Natural Resources for application in a given region.

Regulatory Flood — a flood designated by the Ontario Ministry of Natural Resources for flood plain management purposes.

Relief Flow — the flood flow which bypasses the main structure at a stream crossing by flowing over the roadway or through a relief bridge or culvert.

Return Period — the average period in years between occurrences of a discharge equalling or exceeding a given value.

Revetment — a vertical or inclined facing of rip-rap or other material protecting a soil surface from erosion.

Roadway — the portion of a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.

Scour — local lowering of a streambed by the erosive action of flowing water.

Scoured Depth — the total depth of water measured between design high water level and the scoured streambed.

Sealed Deck Joint — a deck joint designed to prevent the passage of water through the joint.

Shoulder — the portion of a roadway contiguous to the through lanes or auxiliary lanes for accommodating stopped vehicles or providing clearance from sidewalks, bikeways or from the bridge structure.

Sidewalk — a pedestrian walkway located at the edge of a bridge deck.

Soffit — the undersurface of the superstructure of a bridge or culvert.

Speed Change Lane — an auxiliary lane provided for vehicle acceleration or deceleration.

Spread Footing — a footing which transfers loads directly to the underlying foundation material.

Stage — the height of a water surface above a specified datum.

Through Lane — a traffic lane for through traffic movement.

Traffic Lane — a through lane or auxiliary lane for the movement of a single line of vehicles.

Travelled Way — the portion of a roadway for the movement of vehicles, excluding the auxiliary lanes and shoulders.

Notation 1 — 3

C_L	local scour coefficient applicable to non-cohesive silt and sand.
C_S	skew coefficient for piers skewed to flow.
d_{sp}	depth of local scour measured below the anticipated level of maximum general scour adjacent to a pier, m.
d_t	total scoured depth measured from design high water level to bottom of scour, m.
S	shoulder width on approach roadway, m.
w	width of pier exposed to the flow; or the width of footing if scour is likely to expose the footing to the flow.

Bridge Deck Criteria 1 — 4

Bridge Deck Cross-Section 1 — 4.1

1 — 4.1.1 Deck Width

1 — 4.1.1.1 Traffic Lanes

The number and width of lanes on bridges shall correspond with those of the approach roadway. The bridge design shall take into consideration planned future changes in the number or width of lanes.

1 — 4.1.1.2 Single Lane Bridges

Provision of single lane bridges shall be subject to approval. The minimum width between curbs, railings, or curb and railing shall not be less than 5.0 m unless otherwise approved.

1 — 4.1.2 Bridge Deck Shoulder Widths

1 — 4.1.2.1 Except as specified in Clauses 1 — 4.1.2.2 to 1 — 4.1.2.6, shoulder widths on a bridge without sidewalks or bikeways shall be 0.5 m narrower than shoulder widths on the approach roadway, but shall not be less than 1.0 m.

1 — 4.1.2.2 For a bridge without sidewalks or bikeways, and where the approach roadway is provided with continuous barrier walls or curbs, the full approach shoulder widths shall be carried across the bridge.

1 — 4.1.2.3 The shoulder width adjacent to a speed change lane on a bridge without sidewalks or bikeways shall be 1.0 m.

1 — 4.1.2.4 Bridge shoulders shall be widened to not more than 3.0 m where necessary to meet minimum stopping sight distance requirements.

1 — 4.1.2.5 The widths of shoulders on bridges shall be sufficient to accommodate planned future sidewalks or bikeways as shown in Figure 1 — 4.1.3.

1 — 4.1.2.6 Shoulder Widths for Short and Long Bridges

Shoulder widths on short bridges with deck lengths of less than 10 m may be increased, and on long bridges with deck length greater than 200 m may be decreased; such changes shall be subject to approval.

1 — 4.1.2.7 The widths of shoulders adjacent to sidewalks or bikeways on bridges shall be in accordance with Figure 1 — 4.1.3.

1 — 4.1.3 Sidewalk and Bikeway Widths

Widths of sidewalks and bikeways on bridge decks shall be as shown in Figure 1 — 4.1.3, and shall comply with the following requirements.

a) The edge of a sidewalk adjacent to the roadway on a bridge shall match that of the approach sidewalk.

b Where the approach roadway is not provided with a curb, the sidewalk width shall be at least 1.5 m.

c) The clearance from the travelled portion of a bikeway to a curb or other raised object shall be 0.25 m.

1 — 4.1.4 **Width of Bridge Deck Median**

The width of median on a bridge shall match that of the approach roadway.

Curbs **1 — 4.2**

The height of curbs shall be not less than 150 mm above the adjacent roadway except to match the height of curbs on the approach roadway.

Curbs shall not be used in conjunction with barrier walls except at the edge of a sidewalk adjoining the wall.

Sidewalk Height and Crossfall **1 — 4.3**

The height of sidewalk on a bridge shall match the height of sidewalk on the approach roadway. Where there is no sidewalk on the approach roadway, the sidewalk on the bridge shall be at least 150 mm above the adjacent roadway surface.

Figure **Bridge Deck Cross-Section at Sidewalk** **1 — 4.1.3(a)**

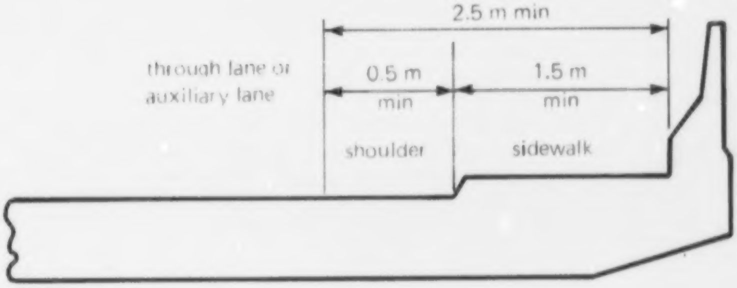

2.5 m min
through lane or auxiliary lane
0.5 m min
1.5 m min
shoulder
sidewalk

Figure **Bridge Deck Cross-Section at Bikeway** **1 — 4.1.3(b)**

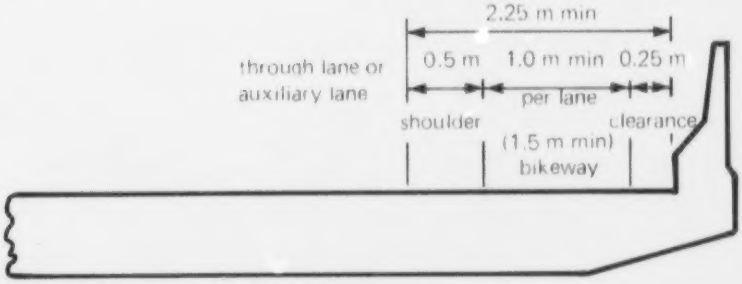
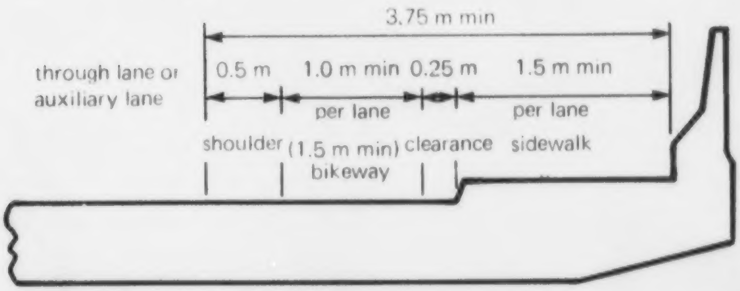

2.25 m min
through lane or auxiliary lane
0.5 m
1.0 m min per lane
0.25 m
shoulder
clearance
(1.5 m min) bikeway

Figure **Bridge Deck Cross-Section at Sidewalk and Bikeway** **1 — 4.1.3(c)**


3.75 m min
through lane or auxiliary lane
0.5 m
1.0 m min per lane
0.25 m
1.5 m min per lane
shoulder
(1.5 m min) bikeway
clearance
sidewalk

Sidewalks shall be sloped down toward the roadway with a slope of 2%.

Roadway Superelevation and Crossfall **1 — 4.4**

The roadway surface shall be superelevated in accordance with standards for the approach roadway. The superelevation shall not exceed 6%, except that on ramps in urban areas with a high standard of winter maintenance, a maximum superelevation of 8% may be permitted.

Roadways and bikeways shall have a minimum crossfall of 2% except for superelevation runouts and wood decks.

Roadway Surface **1 — 4.5**

The riding surface of a bridge deck roadway of material other than wood shall have skid resistance characteristics at least equal to those of the approach roadway.

Longitudinal Deck Profile **1 — 4.6**

There shall be no discontinuity in the longitudinal profile of the bridge deck and its approaches.

Bridge Deck Drainage **1 — 4.7**

1 — 4.7.1 **General**

Transverse drainage of the deck shall be achieved by providing crossfall or superelevation as specified in Clauses 1 — 4.3 and 1 — 4.4. Longitudinal drainage shall be achieved by providing a minimum gradient of 0.5% or by a vertical curve.

Water in the approach roadway gutter shall be intercepted by catch basins or other suitable means to prevent flow onto the bridge deck.

1 — 4.7.2 **Size and Number of Drains**

The size and number of deck drains shall be such that during a 10-year design storm, the gutter flow will not encroach more than 1.5 m on the traffic lanes. Deck drains shall be provided only where a definite need can be established.

1 — 4.7.3 **Bridges Without Deck Drains**

Where deck drains are not used, water flowing from the bridge deck shall be intercepted immediately beyond the end of the bridge or approach slab by catch basins or other suitable means where necessary to prevent erosion of the embankment slopes.

1 — 4.7.4 **Downspouts and Downpipe Systems**

1 — 4.7.4.1 **Location and Length of Downspouts**

The location and length of downspouts shall be such that water will not be discharged against any part of the structure, assuming that the water will spread from the outlet at an angle of 45° from the vertical.

1 — 4.7.4.2 **Projection Below Superstructure**

Downspouts shall not project more than 100 mm below the soffit of the lowest adjacent superstructure component unless a greater length is required by Clause 1 — 4.7.4.1.

1 — 4.7.4.3 **Downpipe Systems**

Downpipe systems for bridge deck drainage shall be used only where no acceptable alternative is available. Downpipes shall be of rigid corrosion-resistant material and shall have an internal width or diameter of at least 200 mm and a slope of at least 8.0%. Elbows no sharper than 45° shall be used, and changes of direction shall have a radius not less than 450 mm. Cleanouts shall be provided in sufficient numbers and at convenient locations to permit easy access to all parts of the downpipe system.

1 — 4.7.5 **Discharge from Deck Drains**

The location and configuration of deck drains shall be such that water will not be discharged onto roadways, railroad rights-of-way, sidewalks or cycle paths, and will not cause erosion.

Surface water from the bridge deck or road surface shall be directed away from the bridge substructure.

1 — 4.7.6 **Drip Grooves**

Continuous drip grooves shall be provided along the underside of a concrete deck at a distance not exceeding 300 mm from the edges.

Maintenance and Inspection Requirements **1 — 4.8**

1 — 4.8.1 **General**

Bridge abutments, piers and decks shall be detailed to facilitate visual inspection and maintenance and replacement of bridge bearings and deck joints.

1 — 4.8.2 **Deck Details at Joints**

A minimum open gap of 200 mm shall be provided below deck joints between the deck and the ballast wall in the case of concrete slab bridges, between the ends of beams and the ballast wall in the case of beam and slab bridges, and between the beam or deck slab ends in the case of deck joints over piers or at the ends of suspended spans.

1 — 4.8.3 **Bridge Bearing Seats**

Where the deck is interrupted by a sealed deck joint, all top surfaces of piers and abutments other than bearing seats shall have a minimum slope of 5% toward their edges.

For open deck joints this minimum slope shall be increased to 3 horizontal to 1 vertical, measured perpendicular to the centreline of bearings.

In the cases of open deck joints, salt and debris shall be prevented from coming into contact with the bearings.

1 — 4.8.4 **Provision for Jacking**

Provision for jacking the superstructure shall be made for all bridges having steel or concrete superstructures. Unfactored dead load reactions and permissible locations for jacking of the superstructure shall be shown on the plans.

1 — 4.8.5 **Access to Box Girders**

Access openings shall be provided for inspection and maintenance of steel and concrete box sections having an interior cross-sectional area of 0.7 m^2 or more.

Access openings shall be provided for each box or cell, and shall have closely fitting hinged covers.

Deck-Supported Utilities **1 — 4.9**

Where required, provision shall be made for accommodating utilities such as electrical conduits, telephone conduits, water pipes, sanitary sewers, light poles and transit wire support poles. Sewers and water lines shall not be placed within structural components.

Gas or oil pipes shall not be carried on or in bridges unless approved.

Approach Slabs **1 — 4.10**

Bridges on paved roads shall be provided with approach slabs anchored to the abutments. The length of approach slabs shall be 6.0 m unless otherwise approved.

Concrete Bridge Deck Protection **1 — 4.11**

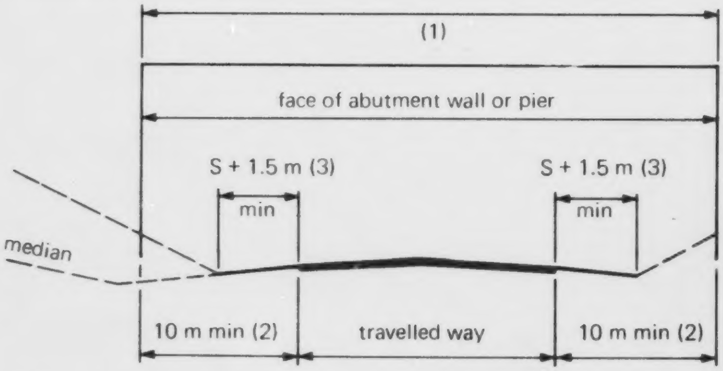
Approved protection against corrosion spalling of concrete bridge decks shall be provided.

Bridge Clearances 1 — 5

Horizontal Clearances for Roadways Under Bridges 1 — 5.1

1 — 5.1.1 Freeways

Figure **Horizontal Clearances for Freeways under Bridges** **1 — 5.1.1**



Notes:

1) To be at least 20 m greater than width of travelled way unless modified by (2).
2) Measured from edge of travelled way. May be reduced to 4.5 m for pier only, subject to approval.
3) Measured from edge of travelled way, or from auxiliary lane if present. May be reduced to S + 0.5 m where a future traffic lane can be accommodated in the median.

1 — 5.1.1.1 **General**

Except as specified in Clauses 1 — 5.1.1.2 and 1 — 5.1.1.3, horizontal clearances for freeways passing under bridges shall be as shown in Figure 1 — 5.1.1.

1 — 5.1.1.2 **Clearance to Median Pier**

Where the median width is less than 20 m plus the width of pier, the pier shall be placed as close to the centreline of the median as practicable and a guiderail or barrier wall provided if the clearance is less than that specified in Figure 1 — 5.1.1. Clearances to guiderails and median barrier walls shall be as shown in Figure 1 — 5.1.2.

1 — 5.1.1.3 A guiderail or barrier wall shall be placed between the abutment or pier and the shoulder where geometric considerations make the clearances shown in Figure 1 — 5.1.1 impracticable. Clearances to guiderails and barrier walls shall be as shown in Figure 1 — 5.1.2.

1 — 5.1.2 **Roads Other Than Freeways**

1 — 5.1.2.1 **General**

Except as specified in Clauses 1 — 5.1.2.2 to 1 — 5.1.2.4, horizontal clearances for roads other than freeways shall be as shown in Figure 1 — 5.1.2 unless otherwise approved.

1 — 5.1.2.2 **Sidewalks and Bikeways**

Widths and horizontal clearances of sidewalks and bikeways shall be as shown in Figure 1 — 5.1.2.2, subject to the following provisions.

a) Curb lines on the bridge shall correspond with those of the approach roadway.
b) Where the approach roadway is not provided with a curb, the sidewalk width shall be at least 1.5 m.
c) Horizontal clearance from the travelled portion of a bikeway to a curb or other raised object shall be 0.25 m.
d) Where a sidewalk is more than 250 mm higher than the roadway, the horizontal clearance from an edge of the through lane or auxiliary lane to the face of sidewalk shall be at least 1.0 m.

1 — 5.1.2.3 **Future Sidewalks and Bikeways**

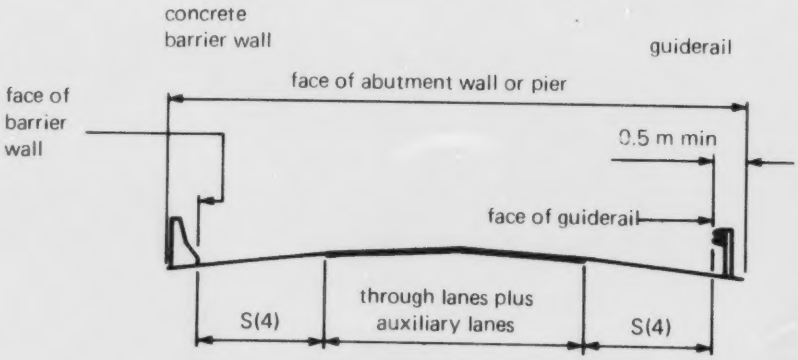
Additional horizontal clearance shall be provided under bridges to accommodate planned future widening of the roadway and the planned future addition of sidewalks or bikeways.

Vertical Clearances for Bridges 1 — 5.2

1 — 5.2.1 **Over Roadways**

Minimum vertical clearance for road bridges with solid or cellular cast-in-place concrete construction shall be 4.65 m over all lanes and shoulders unless otherwise

Figure **Horizontal Clearances for Roads Other Than Freeways** **1 — 5.1.2**



Note:

4) Measured from edge of travelled way, or from auxiliary lane if present. Minimum 1.5 m; this may be reduced to 0.5 m at median piers, subject to approval.

approved. The minimum vertical clearance for all other road bridges shall be 5.0 m over all lanes and shoulders unless otherwise approved.

1 — 5.2.2 **Over Sidewalks and Bikeways**

Minimum vertical clearance for sidewalks and bikeways shall be 2.5 m.

Vertical Clearance Under Pedestrian and Bicycle Bridges **1 — 5.3**

Minimum vertical clearance under pedestrian and bicycle bridges over roadways shall be 5.3 m.

Vertical Clearance to Overhead Sign Supports **1 — 5.4**

Vertical clearance to overhead sign supports shall be as specified in Section 5.

Figure **Horizontal Clearances at Sidewalk Under Bridge** **1 — 5.1.2.2(a)**

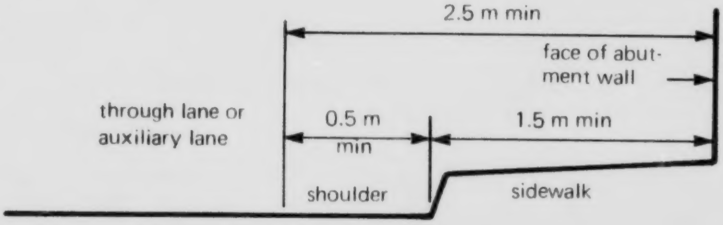

2.5 m min
face of abutment wall
through lane or auxiliary lane
0.5 m min
1.5 m min
shoulder
sidewalk

Figure **Horizontal Clearances at Bikeway Under Bridge** **1 — 5.1.2.2(b)**

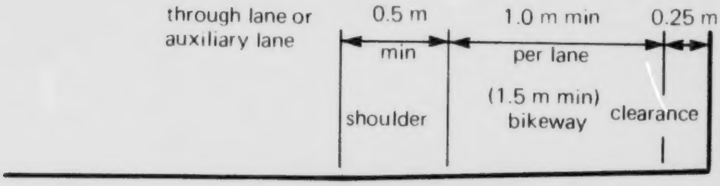
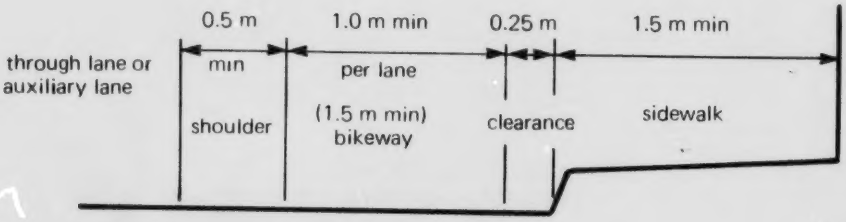

through lane or auxiliary lane
0.5 m min
1.0 m min per lane
0.25 m
shoulder
(1.5 m min) bikeway
clearance

Figure **Horizontal Clearances at Sidewalk and Bikeway Under Bridge** **1 — 5.1.2.2(c)**


0.5 m min
1.0 m min per lane
0.25 m
1.5 m min
through lane or auxiliary lane
shoulder
(1.5 m min) bikeway
clearance
sidewalk

Clearances for Bridges Over Water **1 — 5.5**

1 — 5.5.1 **Hydraulic Clearance**
Hydraulic clearance shall be as specified in Clause 1 — 7.3.5.2.

1 — 5.5.2 **Navigational Clearance**
Navigational clearance shall conform to the requirements of the agency having jurisdiction over the waterway.
The water level used as a basis for measuring navigational clearance shall be the maximum likely to occur during the navigation season.

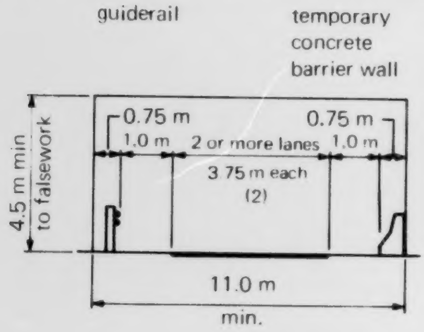
1 — 5.5.3 **Clearances for Pathways Under Bridges Crossing Water**
Minimum width of pedestrian or bicycle paths under bridges crossing water shall be 1.5 m. Minimum vertical clearance shall be 2.5 m.

Clearances for Bridges Over Railways **1 — 5.6**
Clearances for bridges over railways shall conform to the requirements of the Board Order of the Canadian Transport Commission. Allowance shall be made for curvature and superelevation of the track. Temporary clearances during construction shall be as specified by the railway or railways having jurisdiction over the tracks and as set forth in the Board Order of the Canadian Transport Commission.

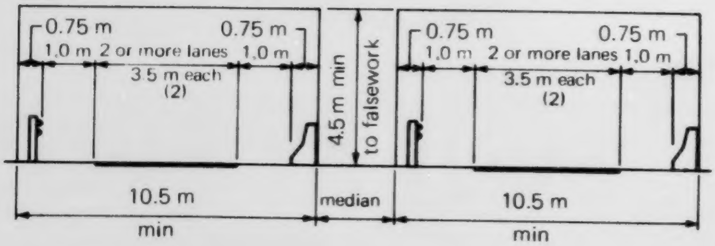
Construction Clearances for Bridges Over Roadways **1 — 5.7**
Construction clearances shall conform to the requirements of the agency having jurisdiction over the roadway. Where no special requirements are imposed the clearances shown in Figure 1 — 5.7 shall apply.

Figure **Construction Clearances for Bridges Over Roadways** **1 — 5.7**

Undivided Highway With One or More Lanes in Each Direction


guiderail
temporary
concrete
barrier wall
4.5 m min
to falsework
0.75 m
0.75 m
1.0 m
2 or more lanes
1.0 m
3.75 m each
(2)
11.0 m
min.

Divided Highway With Two or More Lanes in Each Direction


0.75 m
0.75 m
1.0 m
2 or more lanes
1.0 m
3.5 m each
(2)
4.5 m min
to falsework
0.75 m
0.75 m
1.0 m
2 or more lanes
1.0 m
3.5 m each
(2)
10.5 m
min
median
10.5 m
min

Notes:

1) All horizontal dimensions are minimum.
2) Not to exceed total width of existing through lanes plus auxiliary lanes.

Bridge Railings **1 — 6**

General Requirements **1 — 6.1**
Railings shall be provided on all bridges, and unless specified otherwise by the owner, shall meet the requirements of Clauses 1 — 6.2 and 1 — 6.3 and Section 5.

Combination Railings **1 — 6.2**
Unless pedestrian railings are required by Clause 1 — 6.3.1 the railings shall be combination railings as illustrated in Figure 1 — 6.2. Other configurations may be used provided that they meet the requirements of this Code.

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1 — 6.2.1 **Height of Barrier Walls**

The solid portion of a barrier wall shall be not less than 700 mm high measured from the reference surface as defined in Figure 1 — 6.2, except that when designed with a sloping safety profile the height shall be at least 825 mm.

The metal rail or rails may be omitted provided that the height of the barrier wall is not less than the minimum overall height shown.

1 — 6.2.2 **Geometry of Railing Elements**

a) The centreline of the lowest traffic rail shall be between 375 mm and 500 mm above the reference surface, and that of the highest traffic rail shall not be more than 850 mm above the reference surface. The top of the top rail shall not be less than 1050 mm above the reference surface.

b) The traffic faces of all traffic rails shall be within 25 mm of a vertical plane through the traffic face of the rail closest to traffic. Rails not conforming to this requirement shall not be considered as traffic rails but may be considered in determining the maximum clear vertical opening between rails.

c) Where a sidewalk is provided, the configuration of railing elements shall be such that there is no opening through which a sphere larger than 150 mm in diameter can pass.

1 — 6.2.3 **Urban Freeway Bridges**

Combination railings separating vehicular traffic from sidewalks or bikeways on urban freeway bridges shall be of the barrier wall type with sloping safety profile. The minimum height of the wall portion of the railing shall be 1000 mm measured from the top of the roadway.

1 — 6.2.4 **Transitions to Railings**

Exposed posts and rail ends and sharp changes in the geometry of the railing shall be avoided. A smooth transition consisting of a continuation of the bridge barrier, a guiderail anchored to the bridge end, or other effective means, shall be provided in order to protect traffic from direct collision with the ends of railings.

Pedestrian Railings **1 — 6.3**

1 — 6.3.1 **General**

Pedestrian railings shall be provided at the edges of bridges for pedestrians or bicycles, and at the outer edges of sidewalks or bikeways separated from vehicular traffic by a combination railing.

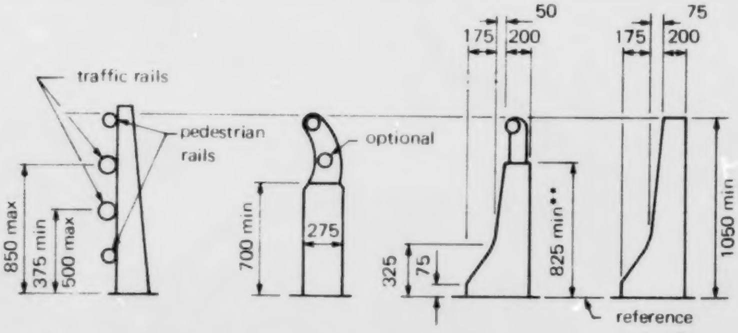
1 — 6.3.2 **Height of Pedestrian Railings**

The minimum height of pedestrian railings, measured from the top of the sidewalk or bikeway surface to the top of the top rail, shall be 1050 mm.

1 — 6.3.3 **Clear Openings in Railings**

The configuration of railing elements shall be such that there is no opening through which a sphere larger than 150 mm in diameter can pass.

Figure **Combination Railings** **1 — 6.2**



All dimensions in millimetres.

*Reference Surface is defined as:

1) Top of sidewalk or bikeway adjacent to railing.
2) Top of roadway adjacent to railing where there is no sidewalk or bikeway.
3) Where railing separates roadway from sidewalk or bikeway:
a) top of adjacent roadway for measuring height of traffic rails and solid portion of barrier walls, and
b) top of adjacent roadway, sidewalk or bikeway, whichever is higher, for measuring overall height of railing.

**1000 min for combination railing separating vehicular traffic from sidewalks or bikeways on urban freeway bridges.

Hydraulic Design — 1 — 7

Field Investigations — 1 — 7.1

Field investigations shall be carried out at all stream crossings, taking into consideration:

a) the hydraulic characteristics of the proposed crossing, including past and potential future problems at and adjacent to the crossing;
b) the hydraulic performance of existing structures on the stream; and
c) other features pertinent to the hydraulic design of the structure.

Design Floods — 1 — 7.2

Design flood criteria shall conform to Provincial Highways Directive B-100.

1 — 7.2.1 Design Flood Discharge

Design flood discharges shall be estimated by the methods given in Chapter C of the MTC *Bridge Hydraulics Manual* or Chapter B of the MTC *Drainage Manual*, or by other approved methods.

1 — 7.2.2 Design High Water Level

1 — 7.2.2.1 For Hydraulic Computations

The high water level used as a basis for hydraulic design computations shall be the water level corresponding to the design flood discharge under ice-free conditions.

If the site is subject to abnormal flood stage conditions the design high water level shall also satisfy the following requirements.

a) For scour computations, the water level shall be the lowest likely to coincide with the design flood.
b) For backwater computations, the water level shall be the highest likely to coincide with the design flood

1 — 7.2.2.2 For Establishing Soffit and Approach Grade

The high water level for establishing the minimum elevation of the soffit and approach road grade shall be the higher of the following.

a) The water level corresponding to the design flood discharge under ice-free conditions, modified for abnormal flood stage conditions as specified in Clause 1 — 7.2.2.1(b). For establishing the minimum grade of the approach road, the water level shall be increased by the amount of backwater created by the crossing.
b) The water level caused by ice jams and having a return period comparable to that selected for the design flood.

Hydraulic Design of Bridges — 1 — 7.3

1 — 7.3.1 General

The design of structure waterways shall satisfy the scour and related criteria specified in Clauses 1 — 7.3.2 and 1 — 7.3.3. Where the size of structure opening is governed by backwater considerations, the design shall also satisfy the requirements of Clause 1 — 7.3.4.

1 — 7.3.2 Design for Scour and Related Factors

1 — 7.3.2.1 General

a) The depth of general scour, natural scour, local scour, degradation and artificial deepening of the channel shall be estimated, where applicable, at all structures having erodible inverts.
b) Depths of scour shall be measured below the average original bed elevation.
c) The total depth of scour at a point in a structure opening shall be taken as the greater of:
   i) maximum general scour + scour, or
   ii) maximum general scour + local scour.

1 — 7.3.2.2 High Water Level for Scour Calculations

Scour calculations shall be based on the design high water level determined in accordance with Clause 1 — 7.2.2.1.

1 — 7.3.2.3 Soils Data

Scour calculations shall take into consideration properties of the streambed and subsoil materials to a depth below the estimated maximum scour.

1 — 7.3.2.4 General Scour

a) Average general scour shall be calculated by an approved method.
b) Maximum general scour at any point in a structure waterway shall be determined by redistributing the average general scour as described in the RTAC *Guide to Bridge Hydraulics*, or by other approved methods.

c) On a shifting channel, the maximum general scour shall be assumed to occur at any point across the structure opening.

d) Where ice jams are likely to occur, the possibility of increased scour shall be taken into account.

1 — 7.3.2.5 **Natural Scour**

a) If a bridge opening is of such dimensions as to create neligible general scour, the depth of maximum natural scour at any point in the bridge waterway shall be estimated from field measurements or by methods detailed in the RTAC *Guide to Bridge Hydraulics*.

b) On a shifting channel, the maximum natural scour shall be assumed to occur at any point across the bridge opening.

1 — 7.3.2.6 **Local Scour at Piers**

a) Local scour shall be measured below the anticipated depth of maximum general scour or maximum natural scour, whichever is applicable, adjacent to the pier.

b) The depth of local scour at a pier in non-cohesive silt or sand shall be calculated by the following formula or by other approved methods:

$$d_{sp} = C_L C_S w$$

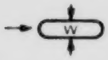
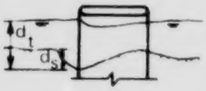
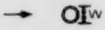
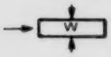
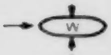
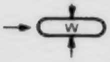
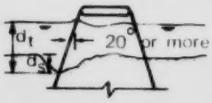
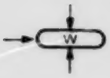
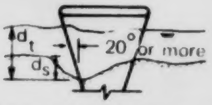
where values of C_L and C_S are given by Tables 1 — 7.3.2(a) and (b), respectively.

If accumulation of driftwood or logs is likely to increase local scour, the width w shall be increased accordingly.

1 — 7.3.2.7 **Degradation**

The average rate of streambed degradation anticipated during the life of a structure shall be estimated from an investigation of existing structures experiencing conditions similar to those at the proposed crossing, or by other approved methods.

Table **Local Scour Coefficients for Piers in Non-Cohesive Silt and Sand and Aligned Parallel to Flow** **1 — 7.3.2(a)**

Pier Shape in Plan*	Pier Shape in Profile	Local Scour Coefficient C_L	
		$d_t \leq 5w$	$d_t > 5w$
		1.5	2.3
	as above	1.5	2.3
	as above	2.0	3.0
	as above	1.2	1.8
		1.0	1.5
		2.0	3.0

* Where the depth of scour is likely to expose the footing to the flow, w shall be taken as the width of the footing.

Table **Skew Coefficent C_s for Piers Skewed at Angle θ to Flow** **1 — 7.3.2(b)**

θ	Length-to-Width Ratio of Pier in Plan		
	4	8	12
0°	1.0	1.0	1.0
15°	1.5	2.0	2.5
30°	2.0	2.5	3.5
45°	2.5	3.5	4.5

1 — 7.3.2.8 Artificial Deepening

The future elevation of the bed of a channel which is likely to be artificially deepened during the life of the structure shall be obtained from drainage plans if available, otherwise it shall be estimated from an investigation of existing structures experiencing similar conditions.

1 — 7.3.3 Protection Against Scour

1 — 7.3.3.1 General

Structure waterways shall be designed so that no movement of any part of the structure will occur as a result of scour or related factors estimated in accordance with Clause 1 — 7.3.2.

1 — 7.3.3.2 Spread Footings

1/ Depth of Footings

a) Where abutments and piers may be exposed to the flow, the bottoms of spread footings shall be located at the lower of the elevations given by (i) to (iii) below, except as specified in (b) to (d) below.

   i) A depth below the average original bed not less than the following.

Abutments except arches	
Spans up to 2.0 m	1.0 m
Spans over 2.0 m and up to 6.0 m	1.2 m
Spans over 6.0 m	1.5 m
Piers and arch abutments	
Spans up to 6.0 m	1.5 m
Spans over 6.0 m	2.0 m
Concrete wingwalls as for abutments	

   ii) A depth below the average original bed not less than 1.7 times the estimated total depth of scour.

   iii) A depth not less than 0.5 m below the level of existing scour.

b) If highly scour-resistant material such as bedrock is encountered at a higher elevation than that given by (a), the footings may be placed at a depth at which they will remain unaffected by scour, degradation or artificial deepening.

c) For piers and abutments constructed of gabions or timber cribs, the depths of footings specified in (a) may be reduced by 50%.

d) If degradation or artificial deepening is expected, the minimum footing depths shall be increased by the amount of the future deepening.

2/ Limitations on Use of Spread Footings

a) Spread footings shall not be founded on sand nor other highly erodible soil unless the bed within the structure opening is paved with concrete or other approved material or the footings are protected against undermining by sheet piles or other approved means.

b) Spread footings immediately adjacent to the stream channel shall not be founded at an elevation higher than the streambed on material other than bedrock or rock fill, unless approved precautions are taken to prevent erosion of the supporting material.

1 — 7.3.3.3 Piles

The use of piles shall be the preferred method of protecting structures against the effects of scour in non-cohesive sands, silts and other highly scourable soils, subject to practical limitations on driving in hard or bouldery materials, and subject also to the following provisions.

a) Penetration — Penetration and structural strength of piles shall be sufficient to ensure their stability under the worst conditions of scour, degradation and artificial deepening.

b) Abutments Supported on Piles — The bottoms of footings of piled abutments exposed to flowing water shall be placed below the maximum expected depth of scour, degradation or artificial deepening, or shall be protected against undermining by sheet piling driven along the front and ends of the footing or by other approved means.

1 — 7.3.3.4 Sheet Piles

Sheet piles used for scour protection shall comply with the following requirements.

a) Sheet piling shall be securely attached to the footings.

b) Penetration and structural strength of sheet piling shall be sufficient to ensure stability of the piling under the worst conditions of scour, degradation and artificial deepening.

1 — 7.3.3.5 Protective Aprons

Aprons used for the protection of piers against local scour shall project from the pier a distance of at least 1.5 times the maximum width of the pier or of footings exposed to the flow.

Rip-rap stone sizes for aprons shall be based on a velocity 1.5 times the average velocity of the design discharge through the structure opening. The thickness of rip-rap aprons shall be 2.0 times the median size of the stone.

1 — 7.3.4 **Design for Backwater**

1 — 7.3.4.1 **General**

The height of backwater shall be computed by an approved method.

1 — 7.3.4.2 **Allowable Backwater**

The backwater created by a structure and its embankments shall comply with the requirements of Provincial Highways Directive B-100.

1 — 7.3.4.3 **Depth of Scour Used in Backwater Calculations**

Unless otherwise approved, the depth of scour assumed in backwater calculations shall be half the average depth of general scour calculated in accordance with Clause 1 — 7.3.2.4.

1 — 7.3.4.4 **Enlargment of Waterway**

Enlargement of the natural channel cross-section at a bridge as a means of reducing backwater shall not be of a nature which will cause erosion of the downstream banks, and the enlargement shall not be liable to be made ineffective by progressive sedimentation or growth of vegetation.

1 — 7.3.4.5 **Reduction of Backwater by Relief Flow**

If use of the regional flood is necessitated by potential flooding of upstream property, the amount of relief flow over the roadway during the regional flood shall be maximized, wherever geometric requirements permit, by using the appropriate minimum freeboard specified in Clause 1 — 7.5.1.

1 — 7.3.5 **Other Hydraulic Requirements for Bridges**

1 — 7.3.5.1 **Location and Alignment**

Structure piers and abutments shall be located and aligned so as to:

a) minimize hindrance to navigation and to the passage of water, ice, debris or logs, as applicable;
b) be compatible with the locations of piers and abutments of adjacent structures;
c) minimize the depth of scour at piers and abutments; and
d) minimize downstream bank erosion.

1 — 7.3.5.2 **Vertical Clearance**

a) Unless otherwise approved, the clearance between the lowest point of the soffit and the design high water level determined in accordance with Clause 1 — 7.2.2.2 shall be sufficient to prevent damage to the structure by the action of flowing water, ice floes or debris, and shall be not less than 1.0 m for freeways, arterial roads and collector roads, and not less than 0.3m for other roads, except as specified in (b) (ii) and (d) below.
b) If use of the regional flood is necessitated by potential flooding of upstream property, the clearance shall comply with the following:
    i) The clearance shall be measured from a water level corresponding to the normal design flood unless hydraulic or geometric requirements dictate otherwise.
    ii) Where relief flow over the roadway is not possible during the regional flood, zero clearance from the regional flood shall be provided unless geometric considerations dictate otherwise.
c) Unless otherwise approved, the clearance between the lowest point of the soffit and normal water level shall not be less than 1.0 m.
d) The requirements of (a), (b) and (c) above shall not apply to structures having curved soffits, the clearances for which shall be based on site-specific conditions and shall be subject to approval.

1 — 7.3.5.3 **Piers and Abutments**

Piers exposed to flowing water shall be pointed or rounded at each end. Timber piles, steel piles or hollow cylinders exposed to impact by heavy ice floes or debris shall be reinforced or braced, or protected by pile clusters, rock-filled cribs or other devices designed to prevent direct impact on the substructure.

Piers and abutments exposed to damage by vessels shall be protected by fenders or other approved devices.

Hydraulic forces on piers shall be determined in accordance with Section 2.

1 — 7.3.5.4 **Buoyancy and Lateral Displacement**

Bridge superstructures having a clearance from design high water level less than the minimum values specified in Clause 1 — 7.3.5.2(a) shall comply with the following requirements:

a) Movement of the superstructure due to buoyancy or the pressure of water, ice or debris shall be prevented.

b) Vents shall be provided in girders or other structural components to prevent the entrapment of air in the event of inundation of the superstructure.

Hydraulic Design of Culverts **1 — 7.4**

1 — 7.4.1 **Closed Invert Culverts**

1 — 7.4.1.1 **General**

Closed invert culverts shall be hydraulically designed in accordance with Chapter D of the MTC *Drainage Manual* or by other approved methods.

Where the capacity of a culvert is controlled by its inlet configuration, consideration shall be given to providing an improved inlet to increase the capacity or permit reduction of the culvert size.

1 — 7.4.1.2 **Allowable Backwater**

The backwater created by a closed invert culvert shall comply with the requirements of Clauses 1 — 7.3.4.2 and 1 — 7.3.4.5.

1 — 7.4.1.3 **Provision for Future Deepening**

The invert of a closed invert culvert on a channel which is likely to be artificially deepened shall be placed at or below the future bed elevation estimated in accordance with Clause 1 — 7.3.2.8.

If the future elevation cannot be predicted, the culvert shall be of the open footing type.

1 — 7.4.2 **End Treatment for Closed Invert Culverts**

1 — 7.4.2.1 **Application**

a) Concrete Box Culverts:
   A cutoff shall be provided at each end of a concrete box culvert.
b) Metal Culverts:
   i) A cutoff, partial headwall, collar or headwall shall be provided at the inlet end of a flexible metal culvert where necessary to protect the culvert against static or dynamic hydraulic forces.
   ii) Embankment slopes shall be modified where necessary to provide sufficient weight of fill to prevent hydraulic uplift of the end of a metal culvert.
   iii) If a weir is provided at the inlet end of a culvert, precautions shall be taken against entry of water into the soil surrounding the culvert.

1 — 7.4.2.2 **Details of End Treatment**

a) Concrete Cutoffs and Headwalls:
   Concrete cutoffs, headwalls and partial headwalls shall extend at least 0.5 m below the estimated limit of scour, subject to the following minimum depth requirements:

Span or Diameter	Minimum Depth Below Culvert Invert
Up to 1.0 m	1.0 m
More than 1.0 m	1.2 m

The minimum depth requirements shall not apply if highly scour-resistant material is encountered at a higher elevation, or if the end treatment is intended primarily as a counterweight against buoyancy forces acting on a metal culvert.

b) Sheet Pile Cutoffs:
   i) Penetration and structural strength of sheet piling shall be sufficient to ensure stability of the piling under maximum scour conditions.
   ii) Sheet piling cutoffs for metal culverts shall have sufficient resistance to prevent upward movement caused by hydraulic uplift forces, taking into account possible vibration of the culvert and loss of adjacent soil due to scour.
c) Attachment to Culvert:
   Cutoffs, headwalls, partial headwalls, collars, end sections and anchorages shall be securely attached to the culvert.
d) Special Requirements for Degrading Streams:
   End treatment for culverts on degrading streams shall comply with the provisions of Clause 1 — 7.6.2.

1 — 7.4.3 **Open-Footing Culverts**

1 — 7.4.3.1 **Hydraulic Design**

a) Erodible Invert:
   An open-footing culvert having an erodible invert shall be designed in accordance with Clauses 1 — 7.3.1 to 1 — 7.3.4. and with Chapter D of the MTC *Drainage Manual*.

b) Inerodible Invert:
An open-footing culvert having an inerodible invert shall be hydraulically designed as a closed invert culvert in accordance with Chapter D of the MTC *Drainage Manual* or by other approved methods.

1 — 7.4.3.2 **Vertical Clearance of Open-Footing Culverts**

Unless otherwise approved, the clearance between the lowest point of a straight soffit and the design high water level determined according to Clause 1 — 7.2.2.2 shall be not less than 0.3 m for spans of 3.0 m or more.

The clearance of a culvert having a curved soffit shall be based on site-specific conditions.

If use of the regional flood is necessitated by potential flooding of upstream property, the clearance shall be in accordance with Clause 1 — 7.3.5.2(b).

1 — 7.4.4 **Other Requirements for Culverts**

1 — 7.4.4.1 **Culvert Extensions**

Extensions to existing culverts shall be designed to avoid the possibility of blockages resulting from changes of direction, cross- sectional shape or number of openings.

1 — 7.4.4.2 **Alignment of Non-Linear Culverts**

Changes of horizontal alignment shall be accomplished by means of gradual curves or by angular changes of direction not exceeding 15° at intervals of not less than 15 m.

1 — 7.4.4.3 **Subdrains for Metal Culverts**

External subdrains shall be provided for metal culverts where considerable groundwater is anticipated.

Freeboard 1 — 7.5

The freeboard of the approach roadway shall be measured from the edge of the travelled way to the design high water level determined in accordance with Clause 1 — 7.2.2.2.

1 — 7.5.1 **Normal Design Flood**

Approach grades shall be set as low as geometric and other considerations permit, but unless otherwise approved, the freeboard shall be not less than 1.0 m for freeways, arterial roads and collector roads, and not less than 0.3 m for other roads.

1 — 7.5.2 **Regional Flood**

If use of the regional flood is necessitated by potential flooding of upstream property, the freeboard shall be based on the water level corresponding to the normal design flood.

1 — 7.5.3 **Freeboard for Pathways Under Bridges Crossing Water**

The freeboard for pedestrian paths or bikeways under bridges crossing water shall be not less than 1.0 m from normal water level for spans of more than 6.0 m, and not less than 0.5 m for spans of 6.0 m or less. These freeboards shall be increased where high maintenance costs are likely to result from use of the minimum values.

Erosion Control 1 — 7.6

1 — 7.6.1 **Bank Protection**

Road embankments and stream channels shall be protected against erosion where necessary to prevent damage to the highway during the design flood and to prevent flow through the structure from damaging adjacent property.

1 — 7.6.2 **Protection Against Degradation**

Culverts at crossings subjected to active streambed degradation shall be provided with special cutoffs, drop structures or other devices, and shall comply with the following requirements:

a) The future depth of degradation shall be estimated in accordance with Clause 1 — 7.3.2.7.
b) Downstream cutoffs shall be of sufficient depth and strength to withstand the amount of degradation expected over a period of at least 20 years.
c) Cutoffs shall be securely attached to the structures.
d) If the invert elevation at the culvert entrance is lower than that of the upstream channel, a sill or wall shall be provided to make up the difference in elevation.

1 — 7.6.3 **Storm Sewer and Channel Outlets**

Outlets of storm sewers and channels discharging into or adjacent to culverts or bridges shall be designed to avoid causing harmful erosion or damage to the structure.

Stream Diversions 1 — 7.7

Stream diversions or other major channel alterations shall be undertaken only if no reasonable alternative is feasible, or at the request of the agency having jurisdiction over the stream.

Requirements of Other Agencies 1 — 7.8

Acceptance of the hydrologic requirements of agencies and others having jurisdiction over a stream shall be subject to approval.

2 Loads

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Reference Publications

Section 2 makes reference to the following publications:

- Special Provisions for Seismic Design, CSA Standard CAN3-A23.3-M77, *Code for the Design of Concrete Structures for Buildings.*
- CSA Standard S269.1-1975, *Falsework for Construction Purposes.*

Scope 2—1

This section specifies loads, load factors and load combinations to be used in calculating load effects for design and for the evaluation of existing structures. Performance factors are given in the appropriate sections of the Code.

Serviceability limit state provisions for the vibration of highway and pedestrian bridges are included.

Provisions concerning construction loads and temporary structures are given which apply to partially completed structures and structures necessary for construction purposes.

Snow loads are not specified, since in normal circumstances, the occurrence of a considerable snow load would cause a compensating reduction in live load.

Definitions 2—2

Barrier — a wall or fence having a vertical surface without openings and reaching to the ground, normally forming a noise barrier or privacy fence.

Controlled Vehicle — a vehicle having known axle weights and dimensions which, when crossing a structure, complies with controls imposed by the structure owner.

Dead Load — the weight of all material supported by the structure and not subject to movement.

Deck Width — the horizontal distance measured at deck level perpendicular to the direction of travel, from face to face of sidewalk, curbs, or barrier walls if curbs are not used.

Design Lane — a longitudinal strip which is a fraction of the deck width, within which a truck or lane load is placed for the purpose of design or evaluation.

Drag — the load in the direction of the approaching wind, induced by an airstream acting on a body.

Ductility — the ability of a structural member to deform after yielding, without failure.

Dynamic Load Allowance — an equivalent static load expressed as a fraction of the Highway Live Loads, which is considered to be equivalent to the dynamic and vibratory effects of the interaction of the moving vehicle and the bridge, including the vehicle response to irregularity in the riding surface.

Effective Temperature — the temperature which governs the thermally induced expansion and contraction of a superstructure.

Exposed Frontal Area — the net area of a body, member or combination of members as seen in elevation; for a superstructure, the sum of the areas of all members including railings and deck system as seen in elevation at 90° to the longitudinal axis in the case of a straight structure or to an axis chosen to maximize wind effects in the case of a structure which is curved in plan.

Exposed Plan Area — the net area of an object as seen in plan from above; in the case of a superstructure, the plan area of the deck and that of any laterally protruding railings, members or attachments.

Factored Load — the product of a load specified in this Code and the corresponding load factor.

Factored Load Effect — the load effect caused by a factored load.

Falsework — a temporary structure used to support another structure or a part thereof.

Flexural Frequency — a natural frequency of vibration of an unloaded bridge based on the longitudinal flexural stiffness and mass distribution of the superstructure.

Gust Effect Coefficient — the ratio of the peak wind-induced load on a structure or response of a structure, including both static and dynamic action, to the static wind-induced load or response; also referred to as the Gust Coefficient.

Ice Accretion — the build-up of an ice layer on the exposed surfaces of a body due to freezing rain or in-cloud icing.

Lane Load — the OHBD Lane Load.

Limit State — a condition beyond which a structure or structural component ceases to fulfill its intended function in an acceptable manner.

Live Load — a load imposed by vehicles, pedestrians, equipment or components subject to movement, other than collision load.

Load — an externally or internally applied load, force, imposed deformation or volumetric change.

Natural Frequency — the inverse of the Natural Period.

Natural Period — the period of free vibration of a single degree-of-freedom structure. For a multi degree-of-freedom system, the natural periods are the periods of the normal modes of vibration.

Normal Mode Shape — the geometric configuration of a structure vibrating at the associated natural frequency.

Pedestrian Bridge — a structure intended primarily for pedestrian use and not loaded by vehicles except for maintenance purposes.

Pedestrian Railing — a railing to contain pedestrians.

Response Spectrum — the envelope of maximum response of a single degree-of-freedom oscillator subjected to a particular disturbance, and plotted as a function of the natural period, or frequency, of the oscillator.

Response Spectrum Analysis — a procedure for computing the response of a structure to a given disturbance for which response spectra have been computed. The total structural response is obtained by combining the contribution of the various modal responses of the structure.

Return Period — the average time in years between the equalling or exceeding of an event; the inverse of the return period is the probability of equalling or exceeding the event in any one year.

Serviceability Limit States Type I — limit states pertaining to fatigue or vibration.

Serviceability Limit States Type II — limit states pertaining to permanent deformation or cracking.

Strouhal Number — a non-dimensional parameter which characterizes the frequency of vortex shedding and represents the ratio of the width of a body placed in an airstream to the wavelength of vortices shed from the body.

Structural Component — a component which influences the strength or stiffness of a structure.

Temporary Structure — a structure required during construction.

Traffic Railing — a railing to contain vehicular traffic.

Travelled Lane — a strip of roadway within which a single line of vehicles is normally expected to travel.

Truck — the OHBD Truck with axles omitted or reduced in weight as and when specified.

Unrestrained Curvature — the curvature induced in an unrestrained member when only temperature effects are considered.

Vortex Shedding — instability of the wake behind a 'bluff', i.e., not streamlined, body in an airstream, comprising a more or less periodic shedding of vortices. The vortices are shed alternately from opposite sides of the body, producing an alternating lateral load normal to the wind direction.

Notation 2—3

A	the area of pier or drift exposed to flowing water, projected parallel to the longitudinal axis of the pier onto a plane perpendicular to that axis, m^2; or the seismic acceleration ratio.
C	the combined seismic response coefficient.
C_A	the ice thickness to pier width aspect ratio coefficient.
C_D	the flowing water drag coefficient.
C_e	the wind exposure coefficient.
C_g	the wind gust effect coefficient.
C_h	the horizontal wind drag coefficient.
C_L	the flowing water load coefficient.
C_N	the coefficient of pier nose inclination from the vertical.
C_v	the vertical wind load coefficient.
D	the diameter of a circular section or the flat-to-flat dimension of a multi-sided section, m.
D_E	the depth of earth cover between the riding surface and the top of a structure, m.
d	the depth of superstructure, m.
F	a framing coefficient; or the horizontal load acting on a pier due to ice pressure, kN.
F_h	the horizontal design pressure due to wind, Pa.
F_v	the vertical design pressure due to wind, Pa.
f_f	the pedestrian footfall frequency, Hz.
g	the acceleration due to gravity, m/s^2.
H	the depth of flowing water at a pier, m; or the height above ground of the top of the superstructure, m.
I	the earthquake resistance importance coefficient.
k_c	a function of r_v.
k_m	the creep maturity coefficient.
k_r	the average creep ratio.
k_t	a function of time, t.
L	the length of a pier, along the longitudinal axis, m.

n	the natural frequency of vibration, Hz; or the number of design lanes on a bridge.
n_e	the frequency of vortex shedding, Hz.
OHBD	Ontario Highway Bridge Design.
P	the total load acting on a pier in the direction of its longitudinal axis, due to flowing water, N; or the total uniformly-distributed load required to cause a 25 mm maximum horizontal deflection of a structure, N.
P_p	the total load acting on a pier in the horizontal direction perpendicular to its longitudinal axis, due to flowing water, N.
p	the pressure exerted by ice on a pier, kPa; or pedestrian loading, kPa.
Q	the equivalent static horizontal earthquake load, N.
Q_R	the design load for earthquake resistance restraining features, N.
q	the reference wind pressure for the design return period, Pa; or uniformly-distributed load, kN/m.
R	radius of curvature, m.
r	radius of curve, m.
r_v	the volume per unit length of a concrete section divided by the area in contact with freely-moving air, mm.
S	Strouhal number.
s	the total loaded length of walkway, m.
T	the natural period of a structure, s; or the torque on a support, N·m.
t	the thickness of ice in contact with a pier, m; or time after application of load, in days; or time after casting concrete, in days; or the time variable in a footfall impulse function, s.
V	the mean velocity of flowing water, m/s; or the mean wind speed, m/s.
v	the design speed of a highway, km/h.
W	the total dead load weight of a structure engaged in vibration, including asphalt, etc., N.
W_c	the contributing weight, being the dead load weight of that part of a structure which during seismic motion exerts the largest load on a restraining feature, N; or deck width, m.
W_e	width of design lane, m.
W_h	the total normal wind load on exposed horizontal supports, N.
W_l	the total normal wind load on luminaires, N.
W_p	the total normal wind load on a sign panel or traffic signal, N.
W_v	the total normal wind load on exposed vertical supports, N.
w	the frontal width of a pier in contact with ice, where the ice is to be split or crushed, measured perpendicular to the direction of ice motion, m.
α_x	load factor for load x, where x is a letter identifying a load, from the list in Table 2—5.1(a).
θ	the angle of the pier nose from the vertical.
γ	the mass density of water, kg/m³.
ϵ_b	the basic concrete shrinkage strain.
ϵ_e	elastic strain.
ϵ_{cr}	concrete creep strain at time t.
ϵ_{sh}	concrete shrinkage strain at time t.
ΣQ_c	The sum of the column shears occurring as a result of the application of the equivalent static horizontal earthquake load Q, N.

Loads 2—4

General 2—4.1

The following loads shall be considered:

- dead load;
- live loads, including:
  - —highway live loads,
  - —dynamic load allowance,
  - —sidewalk load,
  - —curb load,
  - —railing load,
  - —pedestrian bridge load,
  - —centrifugal force,
  - —braking force,

- rail transit loads, and
- maintenance access loads;

- Other loads, including collision load and loads due to:
  - wind,
  - hydrostatic pressure,
  - ice pressure and stream flow,
  - temperature, shrinkage and creep,
  - earth pressure,
  - earthquake,
  - ice accretion,
  - elastic strain,
  - bearing behaviour,
  - differential foundation settlement, and
  - prestressing.

Approval shall be obtained when special circumstances require the consideration of loads or load magnitudes other than those specified.

Dead Load **2 — 4.2**

2 — 4.2.1 Dead load shall include the weight of all fixed parts of the structure including appendages to the structure, the wearing surface, earth cover and utilities.

2 — 4.2.2 In the absence of more precise information, the unit weights given in Table 2 — 4.2.2 shall be used in computing dead loads.

Table **Unit Material Weights** **2 — 4.2.2**

Material		**Unit Weight kN/m^3**
Aluminum Alloy		27.0
Bituminous Wearing Surface		23.5
Concrete:	Low-density Concrete	19.0
	Plain Concrete	23.5
	Prestressed Concrete	24.5
	Reinforced Concrete	24.0
Clay and Silt		19.0
Coarse-grained Soil		21.0
Crushed Rock		22.0
Fine-grained or Sandy Soil		20.0
Fly Ash		16.0
Glacial Till		22.0
Rockfill		21.0
Slag:	Air-cooled Slag	11.0
	Water-cooled Slag	15.0
Steel		77.0
Wood:	Hardwood	9.5
	Softwood	6.0

2 — 4.2.3 Earth load other than dead load shall be in accordance with the provisions of Section 6, except that the provisions of Section 12 shall apply for soil-steel structures.

2 — 4.2.4 The effects of immersion in water or exposure to water pressure shall be considered provided that such effects, when beneficial, are certain to be always in existence. When not beneficial, the effects of buoyancy shall be included unless the possibility of their occurrence can, with certainty, be excluded. The unit weight of fresh water assumed shall be 9.80 kN/m^3. The unit weight of salt water or heavily polluted water shall be based on tests of representative samples taken at the site.

The weight of water shall be considered as a dead load. Other effects, including lateral or upward water pressure and buoyancy, shall be considered as hydrostatic pressures.

The water levels assumed shall be the maximum or minimum probable, whichever produces the larger response.

Live Load **2 — 4.3**

2 — 4.3.1 **Highway Live Loads**

2 — 4.3.1.1 **Highway Classification**

Highway Classes shall be as specified in Table 2 — 4.3.1.1.

Table **Highway Classification** **2 — 4.3.1.1**

Average Daily Traffic per Lane Known or Projected	Criteria for use only when Traffic Data are not Available	Highway Class
Truck Traffic > 1000 or Traffic > 4000	Roads primarily for through traffic	A
Truck Traffic > 250 and ≤ 1000 or Traffic > 1000 and ≤ 4000	Roads on which traffic movement and property access have similar importance	B
Truck Traffic > 50 and ≤ 250 or Traffic > 100 and ≤ 1000	Roads primarily for property access which carry moderate commercial traffic	C_1
Truck Traffic ≤ 50 or Traffic ≤ 100	Roads primarily for property access which carry little or no commercial traffic	C_2

Figure **OHBD Truck** **2 — 4.3.1.2(a)**

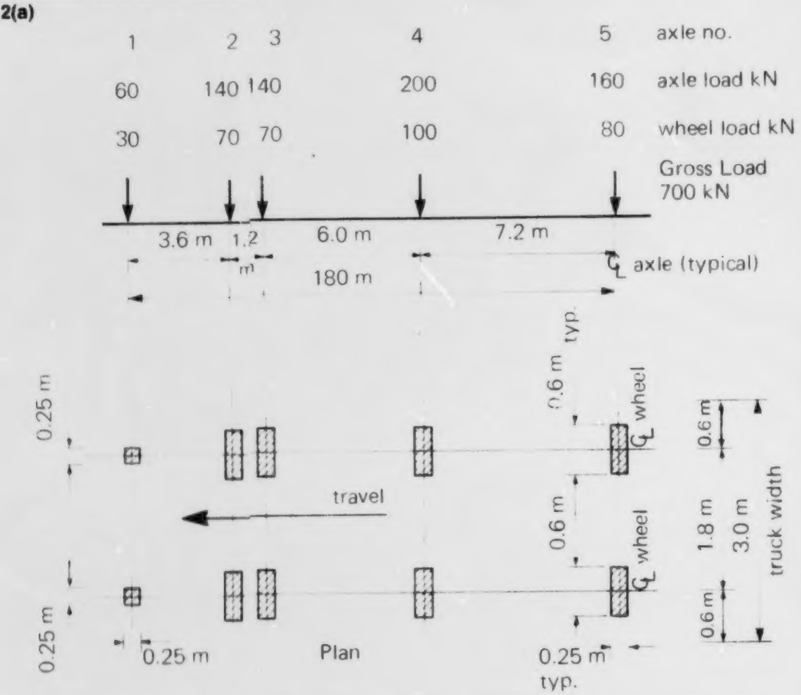
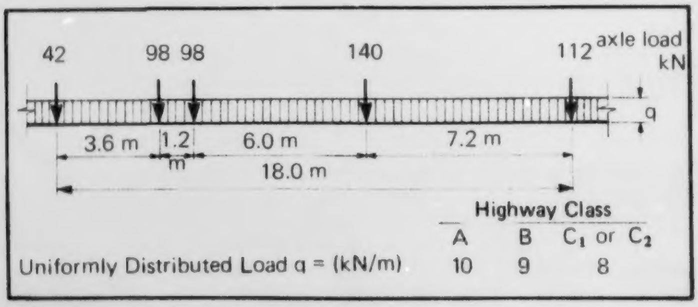

1 2 3 4 5 axle no.
60 140 140 200 160 axle load kN
30 70 70 100 80 wheel load kN
Gross Load 700 kN
3.6 m
1.2 m
6.0 m
7.2 m
180 m
℄ axle (typical)
0.6 m typ.
℄ wheel
0.25 m
travel
0.6 m
℄ wheel
0.6 m
1.8 m
3.0 m
truck width
0.6 m
0.25 m
0.25 m
Plan
0.25 m typ.

Figure **OHBD Lane Load** **2 — 4.3.1.2(b)**


42 98 98 140 112 axle load kN
q
3.6 m
1.2 m
6.0 m
7.2 m
18.0 m

	Highway Class		
	A	B	C_1 or C_2
Uniformly Distributed Load q = (kN/m)	10	9	8

2—4.3.1.2 **OHBD Load**

The OHBD Load shall be the Truck or Lane Load.

a) The OHBD Truck shall be the idealized five-axle truck shown in Figure 2—4.3.1.2(a).

b) The OHBD Lane Load shall consist of an OHBD Truck with each axle load reduced to 70%, superimposed centrally within the width of a 3.0 m wide uniformly-distributed load of magnitude q, as given in Figure 2—4.3.1.2(b) for the Class of Highway supported by the structure.

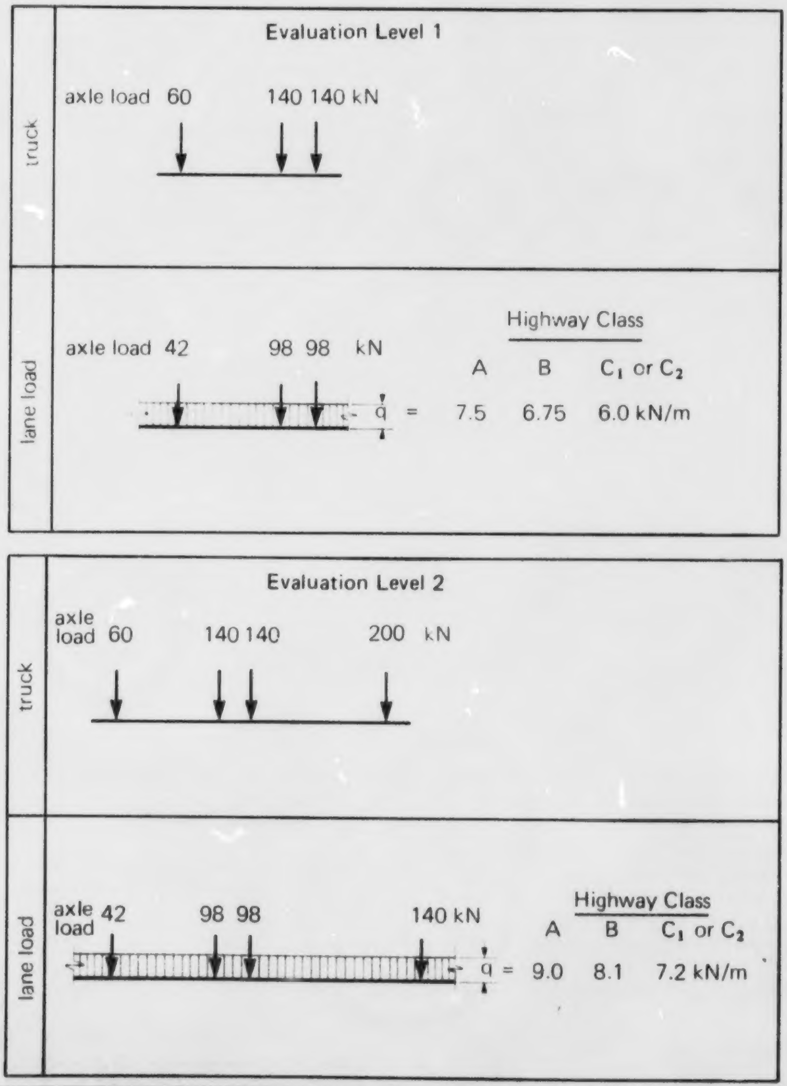
2—4.3.1.3 **Ontario Highway Bridge Evaluation Loads**

a) For Evaluation Level 1, the truck and lane load shall be the OHBD Truck and the OHBD Lane Load without the fourth and fifth axles of the Truck and with the uniformly-distributed load reduced to 75%, as shown in Figure 2—4.3.1.3.

b) For Evaluation Level 2, the truck and lane load shall be the OHBD Truck and the OHBD Lane Load without the fifth axle of the Truck and with the uniformly-distributed load reduced to 90%, as shown in Figure 2—4.3.1.3.

c) For Evaluation Level 3, the OHBD Load shall be used.

Figure **Ontario Highway Bridge Evaluation Load** **2—4.3.1.3**


Evaluation Level 1
truck
axle load 60 140 140 kN
lane load
axle load 42 98 98 kN
Highway Class
A B C₁ or C₂
q = 7.5 6.75 6.0 kN/m
Evaluation Level 2
truck
axle load 60 140 140 200 kN
lane load
axle load 42 98 98 140 kN
Highway Class
A B C₁ or C₂
q = 9.0 8.1 7.2 kN/m

d) Evaluation for controlled vehicles shall be carried out using actual wheel loads and dimensions. The assumed position and direction of the vehicle and the simultaneous application of other live loads shall be in accordance with the controls imposed.

2—4.3.1.4 **Design Lanes**

The number of design lanes shall be determined from Table 2—4.3.1.4. Each design lane shall have a width of $W_e = W_c/n$.

Table **Number of Design Lanes** **2—4.3.1.4**

W_c		n
less than	6.0 m	1
	6.0 m to 10.0 m incl.	2
over	10.0 m to 13.5 m incl.	3
over	13.5 m to 17.0 m incl.	4
over	17.0 m to 20.5 m incl.	5
over	20.5 m to 24.0 m incl.	6
over	24.0 m to 27.5 m incl.	7
over	27.5 m	8

2—4.3.1.5 Truck axle loads that reduce the load effect shall be neglected.

2—4.3.1.6 The uniformly-distributed load shall be applied only where its application increases the load effect.

2—4.3.1.7 For serviceability limit states Type I, the highway live load shall be one Truck only, placed at the centre of a travelled lane. Only one lane shall be loaded. The Lane Load shall not be considered.

2—4.3.1.8 For serviceability limit states Type II and for the ultimate limit states, the highway live load shall be the Truck or the Lane Load, whichever produces the maximum load effect.

The Truck or Lane Load shall be assumed to occupy any position within each design lane, with the direction of travel in either direction parallel to the design lane. The Truck width shall not project beyond the design lane except as specified in Clauses 2—4.3.1.10 and 2—4.3.1.11.

2—4.3.1.9 When more than one design lane is loaded, the highway live loads shall be multiplied by a modification factor obtained from Table 2—4.3.1.9. The design lanes to be loaded shall be selected so as to maximize the load effect.

Table **Modification Factors for Multi-Lane Loading** **2—4.3.1.9**

Number of Loaded Design Lanes	**Modification Factor**
1	1.00
2	0.90
3	0.80
4	0.70
5	0.60
6 or more	0.55

2—4.3.1.10 In computing local responses in deck components, the second and third Truck axles shall be considered with their wheel centres not less than 0.3 m from the face of the curb, railing or barrier wall. This requirement shall not apply to longitudinal effects in slab bridges, nor to main girders, and shall apply only at the ultimate limit states.

2—4.3.1.11 When sidewalks and other areas adjacent to roadways are separated from them by curbs and not by traffic railings, local responses shall be computed by considering an OHBD Truck with each axle load reduced to 70% and with its wheel centres not less than 0.3 m from the face of the railing. This requirement shall not apply to longitudinal effects in slab bridges, nor to main girders, and shall apply only at the ultimate limit states.

2—4.3.2 **Dynamic Load Allowance**

2—4.3.2.1 Highway live loads including controlled vehicle loads, shall be increased by the dynamic load allowance unless otherwise specified.

2—4.3.2.2 The values specified for the dynamic load allowance shall apply unless alternative values based on tests or dynamic analysis are approved.

2—4.3.2.3 For a single axle or single wheel load, with no other highway live load considered in the design lane, the dynamic load allowance shall be 0.40.

2—4.3.2.4 For loading by any two or more axle loads of spans greater than 22 m in length, and spans continuous therewith, the dynamic load allowance shall be obtained from Figure 2—4.3.2.4. In the calculation of the first flexural frequency, the static values of material properties shall be used.

2—4.3.2.5 The dynamic load allowance for the uniformly-distributed portion of the Lane Load shall be 0.10.

2—4.3.2.6 For soil-steel structures, the dynamic load allowance shall be 0.4, reduced according to Clause 2—4.3.2.9.

2—4.3.2.7 The dynamic load allowance shall be 0.30 unless otherwise specified in Clauses 2—4.3.2.3 to 2—4.3.2.6. This clause shall apply for loading by other than a single wheel or single axle of all transverse members and spans, all simple spans not greater that 22 m in length, and all superstructures composed of continuous spans with no span length greater than 22 m.

2—4.3.2.8 The dynamic load allowance shall be included as part of loads transferred from the superstructure to the substructure, but shall not be included in loads transferred to footings that are surrounded with earth nor to those parts of piles that are below ground.

2—4.3.2.9 When there is a depth of earth cover, D_E, between the riding surface and the top of the structure, the dynamic load allowance shall be multiplied by the reduction factor $(1 - 0.5D_E)$. After this reduction, the dynamic load allowance shall not be less than 0.10. For arch and soil steel structures, the fill depth shall be measured at the crown.

2—4.3.2.10 For wood components, the dynamic load allowance resulting from Clauses 2—4.3.2.3 to 2—4.3.2.9 shall be multiplied by 0.70.

2—4.3.2.11 In evaluation, for a single controlled vehicle crossing a structure at a restricted speed and with all other truck traffic excluded from the structure, the dynamic load allowance resulting from Clauses 2—4.3.2.3 to 2—4.3.2.10 shall be multiplied by:

a) 0.30 for a speed limit of 10 km/h or less;
b) 0.50 for speed limits between 10 and 25 km/h;
c) 1.00 for speed limits in excess of 25 km/h.

2—4.3.3 **Sidewalk Load**

2—4.3.3.1 For sidewalks on a highway bridge, the pedestrian live load applied to the walkway area shall be (a) or (b) below, whichever produces the maximum load effect. The pedestrian live load shall be considered only at the ultimate limit states.

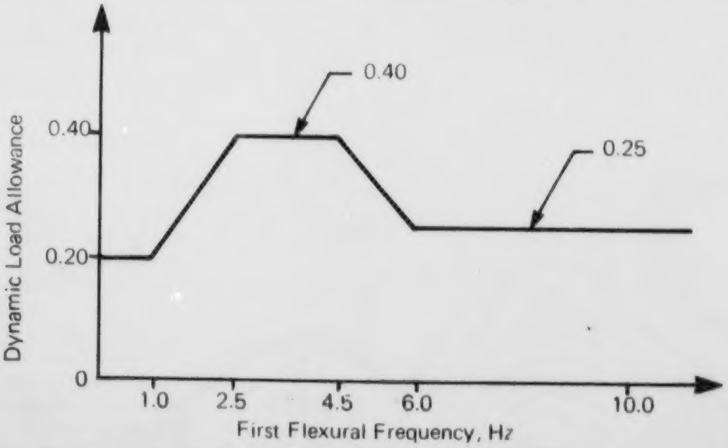
a) A load of 5.0 kPa applied neglecting any contribution to the load effect due to highway live load.
b) When highway live loads in design lanes contribute to the load effect, the load shall be

$$p = 5.0 - s/30$$

where $1.5 \leq p \leq 4.0$ kPa.

2—4.3.3.2 The highway live load specified in Clause 2—4.3.1.11 and the pedestrian live load shall not be considered to act simultaneously on the sidewalk. The maintenance vehicle load specified in Clause 2—4.3.10.3 shall not be considered to act simultaneously with either the pedestrian live load or the highway live load on the sidewalk.

Figure **Dynamic Load Allowance** **2—4.3.2.4**



2—4.3.4 **Curb Load**

2—4.3.4.1 For continuously-supported curbs, the design load shall be a uniformly-distributed lateral load of 25 kN/m.

2—4.3.4.2 For curbs supported at discrete points, the design load shall be a concentrated lateral load of 40 kN.

2—4.3.4.3 Curb loads shall be applied at the top of the curb or 0.25 m above the deck surface, whichever is lower. Curb loads shall be considered only at the ultimate limit states.

2—4.3.5 **Railing Load**

2—4.3.5.1 The pedestrian load on railings shall be a uniformly-distributed lateral load of 1.5 kN/m.

2—4.3.5.2 The traffic load on railings shall be a concentrated lateral load of 80 kN.

2—4.3.5.3 Railing loads shall be applied in accordance with the provisions of Section 5 and only at the ultimate limit states.

2—4.3.5.4 The traffic load on railings shall not be considered to act simultaneously with the curb load.

2—4.3.6 **Pedestrian Bridge Load**

2—4.3.6.1 For pedestrian bridges, the pedestrian live load applied to the walkway area shall be:

$$p = 6.2 - s/25$$

where: $2.0 \leq p \leq 5.0$ kPa

2—4.3.6.2 The pedestrian live load and the maintenance vehicle load specified in Clause 2—4.3.10.3 shall not be considered to act simultaneously.

2—4.3.7 **Centrifugal Force**

For structures on curves, the horizontal centrifugal force shall be computed by multiplying the highway live loads, excluding Lane Load and without dynamic load allowance, by the expression $v^2/127r$, which is to be taken as non-dimensional.

The centrifugal force shall be applied horizontally and at right angles to the direction of travel, 1.8 m above the deck surface at the centre of each design lane.

2—4.3.8 **Braking Force**

2—4.3.8.1 The braking force for a design lane shall be taken as an equivalent static force of either 160 kN or 10% of the Lane Load, whichever is greater. The dynamic load allowance shall not be included.

2—4.3.8.2 The equivalent static braking force shall be applied longitudinally 1.8 m above the deck surface in design lanes intended to carry traffic in the same direction and shall be considered only at the ultimate limit states. No more than two design lanes shall be considered loaded. When two lanes are considered loaded, the equivalent static braking force shall be reduced by 25%.

2—4.3.9 **Rail Transit Loads**

Rail transit loads and load factors shall be as specified by the appropriate transit authority.

2—4.3.10 **Maintenance Access Loads**

2—4.3.10.1 Service walkways, safety gratings and members supporting them shall be designed for a live load of 2.0 kN uniformly distributed over a rectangular area 1.00 × 0.50 m placed anywhere on the walkway or grating.

2—4.3.10.2 Manhole steps and ladder rungs shall be designed for a load of 1.1 kN distributed over a length of 100 mm anywhere on the tread area. The tread area shall include all of the horizontal part of the step or rung except parts beyond bends or other features which effectively limit the usable tread length.

2—4.3.10.3 If the walkway width of a sidewalk or pedestrian bridge is greater than 3.0 m and access for maintenance vehicles is provided, the maintenance vehicle load shown in Figure 2—4.3.10.3 shall be considered on the walkway area.

2—4.3.10.4 Maintenance access loads shall be considered only at the ultimate limit states except that for pedestrian bridges, the maintenance vehicle load shall be considered also at serviceability limit states Type II.

Thermal, Shrinkage, and Creep Effects

2—4.4

2—4.4.1 **General**

Clause 2—4.4 gives requirements for the design of structures to allow for expansion and contraction due to temperature, shrinkage and creep, and for the presence of thermal gradients through the depth of the superstructure. The provisions apply to concrete structures, steel structures, and composite structures built from these materials.

For conventional wood structures, analysis of temperature, shrinkage, and creep effects is not required, but the occurrence of shrinkage and swelling perpendicular to the grain due to moisture changes shall be considered.

The clauses dealing with the shrinkage and creep of concrete are applicable to normal-density concrete and where approximate values are appropriate. For low-density concrete structures, properties established for the actual materials to be used shall be employed.

2—4.4.2 Movements and Load Effects

Provision shall be made for all expansion and contraction that can occur as a result of variations in effective temperature, shrinkage and creep. All load effects induced by any restraint of these movements, including temporary restraints required during construction, shall be included in the analysis.

2—4.4.3 Temperature Effects

2—4.4.3.1 Temperature Range

The range of effective temperature shall be as specified in Table 2—4.4.3.1. The maximum and minimum daily mean temperature shall be as given in Appendix A2—1.

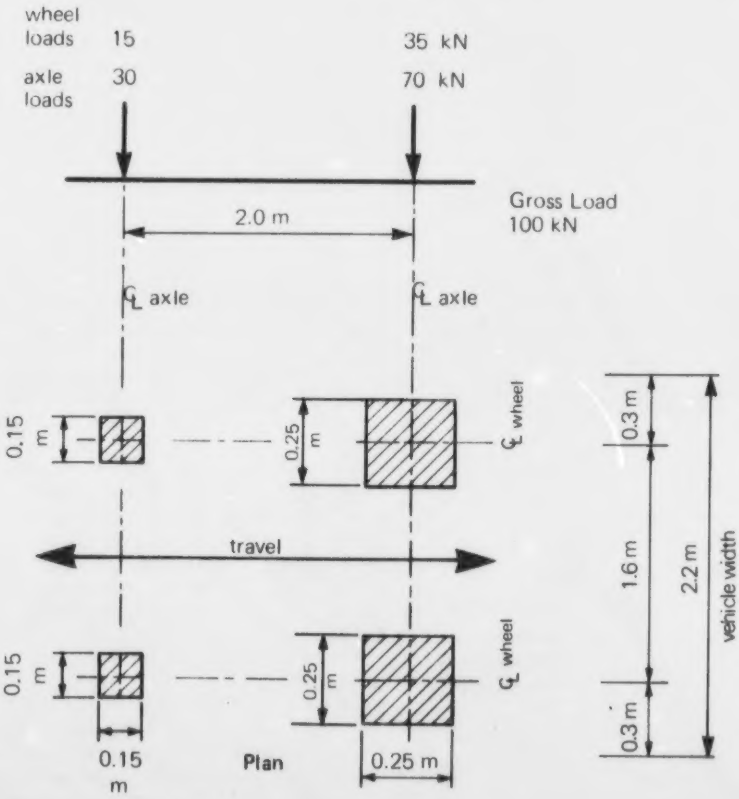
Type A superstructures include steel beam, box, or truss systems with steel decks.

Type B superstructures include steel beam, box, or truss systems with concrete decks.

Type C superstructures include concrete systems with concrete decks.

For Type A, B, and C superstructures, effective temperature is a function of depth. Figure 2—4.4.3.1 gives permissible reductions to the effective temperature ranges specified in Table 2—4.4.3.1.

Figure **Maintenance Vehicle Load** **2—4.3.10.3**



2—4.4.3.2 **Effective Construction Temperature**

In the absence of more reliable data, an effective construction temperature of 15°C shall be assumed for design. This temperature shall be used to determine the effective temperature ranges to be used in the calculation of expansion and contraction.

For Type C structures that are cast-in-place, the heat generated by cement hydration may result in its initial set occurring at above the effective construction temperature. If a more precise method of prediction is not available, it shall be assumed that concrete cools by 25°C from its initial set to the effective construction temperature.

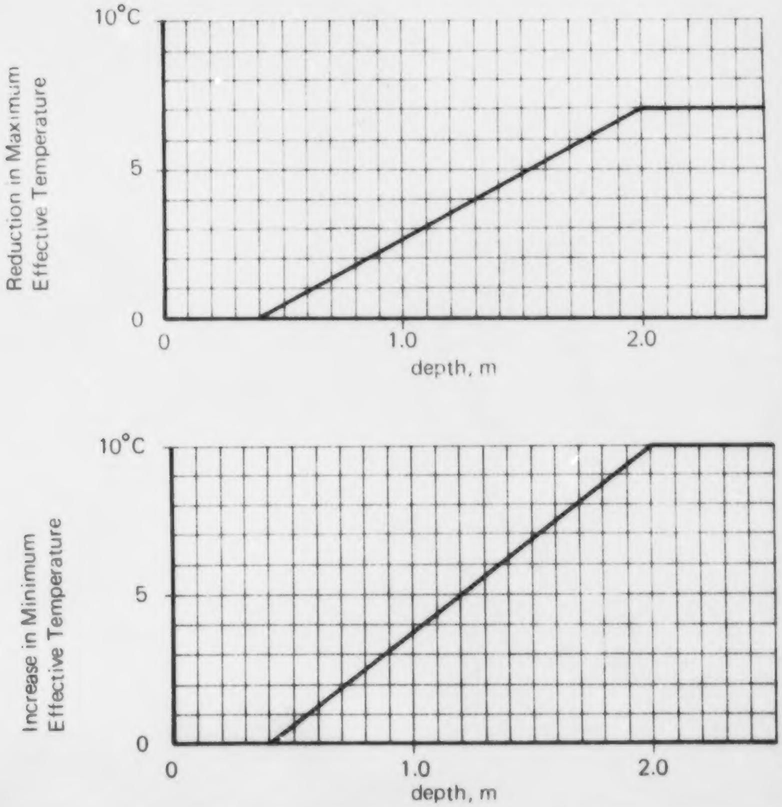
2—4.4.3.3 **Position of Bearings and Expansion Joints**

The plans shall indicate the positioning of bearings and expansion joints in accordance with the requirements of Section 4.

Table **Effective Temperature Range** **2—4.4.3.1**

Superstructure Type	Maximum Effective Temperature	Minimum Effective Temperature
Type A	25°C above maximum daily mean temp.	15°C below minimum daily mean temp.
Type B	20°C above maximum daily mean temp.	5°C below minimum daily mean temp.
Type C	10°C above maximum daily mean temp.	5°C below minimum daily mean temp.

Figure **Permissible Reduction in Effective Temperature Range** **2—4.4.3.1**



2 — 4.4.3.4 Thermal Gradient Effects

The effects of temperature differentials shall be considered in the design of Type A, B and C structures.

A temperature differential is positive when the top surface of the superstructure is warmer than the bottom surface.

Values of unrestrained curvature caused by temperature differentials are given for Type A and Type C structures in Figure 2 — 4.4.3.4. For winter conditions, positive and negative differentials shall be considered. For summer conditions, only positive differentials shall be considered.

For composite and non-composite structures having concrete decks and steel beams, a positive linear temperature differential of 15°C, through the concrete deck only, shall be assumed. Negative differentials need not be considered.

When the coefficients of thermal expansion of the materials used in a composite structure are likely to differ, allowances shall be made for the stresses and movements induced.

2 — 4.4.3.5 Coefficient of Thermal Expansion

The coefficient of linear thermal expansion for steel shall be taken as $12 \times 10^{-6}/°C$. In the absence of more precise information, the coefficient of linear thermal expansion for normal concrete shall be taken as $12 \times 10^{-6}/°C$.

2 — 4.4.4 Shrinkage in Concrete

Provided that concrete aggregates which cause excessive shrinkage are excluded, and curing procedures which prevent excessive early loss of moisture are followed, the shrinkage at time t, ϵ_{sh}, shall be taken as:

$$\epsilon_{sh} = \epsilon_b k_c k_t$$

where: ϵ_b is as specified in Table 2 — 4.4.4;
K_c is as given in Figure 2 — 4.4.4(a);
K_t is as given in Figure 2 — 4.4.4(b) in which t is the time after casting in days.

For serviceability limit states Type II, the difference in free shrinkage between two components of a composite member shall be considered in design. In the absence of more exact data, a differential shrinkage strain of 100×10^{-6} shall be assumed for precast concrete beams with cast-in-place concrete decks. A differential shrinkage strain corresponding to the shrinkage of the concrete shall be assumed for steel beams with concrete decks.

2 — 4.4.5 Creep in Concrete

In the absence of more accurate data, deformations due to creep at constant stress shall be calculated from:

Figure **Unrestrained Curvature for Type A and C Superstructures** **2 — 4.4.3.4**

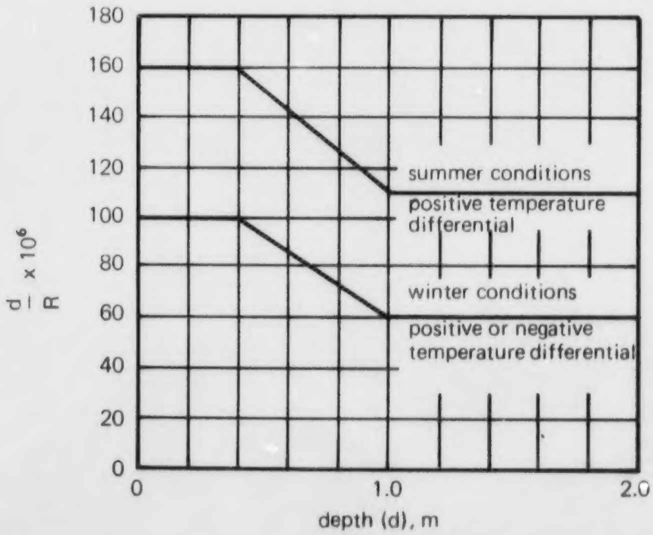


Table **Coefficient ϵ_b** **2 — 4.4.4**

Environment	ϵ_b, Basic Shrinkage Strain × 10^6
Dry air, approximately 50% annual mean relative humidity, as given in Appendix A2 — 1.	520
Open air, not subject to prolonged periods of high temperature and low humidity, approximately 70% annual mean relative humidity, as given in Appendix A2 — 1.	320
Very humid air, e.g., coastal areas, approximately 90% annual mean relative humidity, as given in Appendix A2 — 1.	130
Immersed in water.	0

Figure **Coefficient k_c** **2 — 4.4.4(a)**

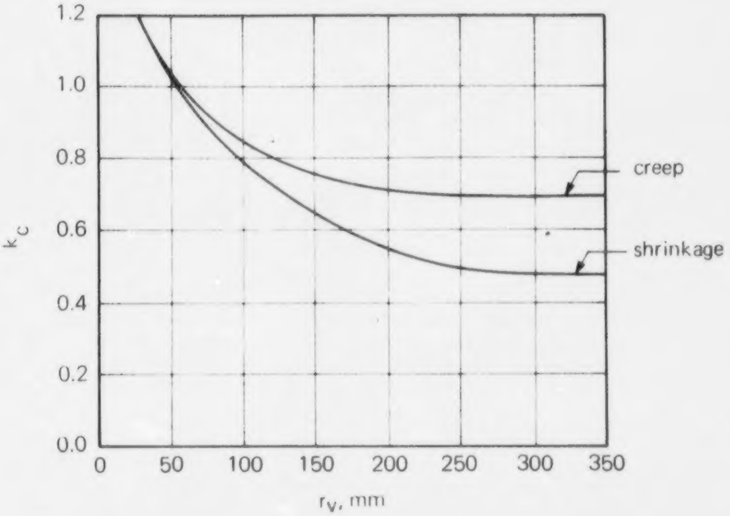
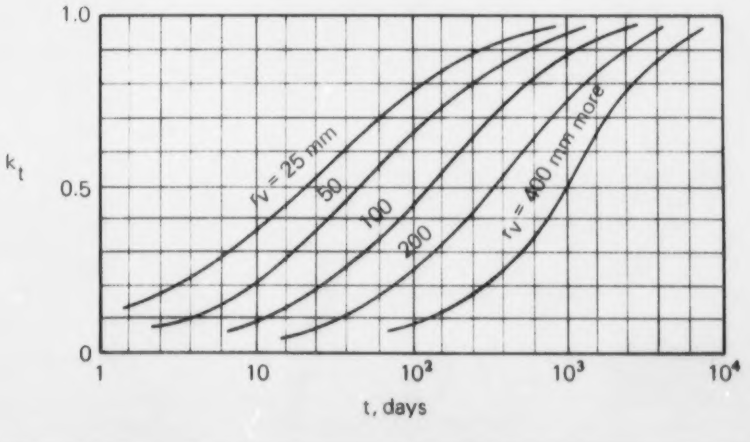


Figure **Coefficient k_t** **2 — 4.4.4(b)**



$$\epsilon_{cr} = \epsilon_e\, k_r\, k_c\, k_t\, k_m$$

where:
ϵ_{cr} = the creep strain at time t;
ϵ_e = the elastic strain;
k_r = the average creep ratio as given in Table 2—4.4.5;
k_c and k_t are as specified in Clause 2—4.4.4, except that t shall be the time in days after application of the load;
k_m is as given in Figure 2—4.4.5.

The beneficial effects of concrete creep in reducing the stresses developed as a result of shrinkage may be considered in the design. In the absence of a more exact procedure, it may be assumed that shrinkage stresses are reduced by 60% as a result of creep.

Table **Coefficient k_r** **2—4.4.5**

Environment	k_r
Dry air, approximately 50% annual mean relative humidity, as given in Appendix A2—1.	3.0
Open air, not subject to prolonged periods of high temperature and low humidity, approximately 70% annual mean relative humidity, as given in Appendix A2—1	2.5
Very humid air, e.g., coastal areas, approximately 90% annual mean relative humidity, as given in Appendix A2—1.	1.5
Immersed in water.	1.0

Wind Loads **2—4.5**

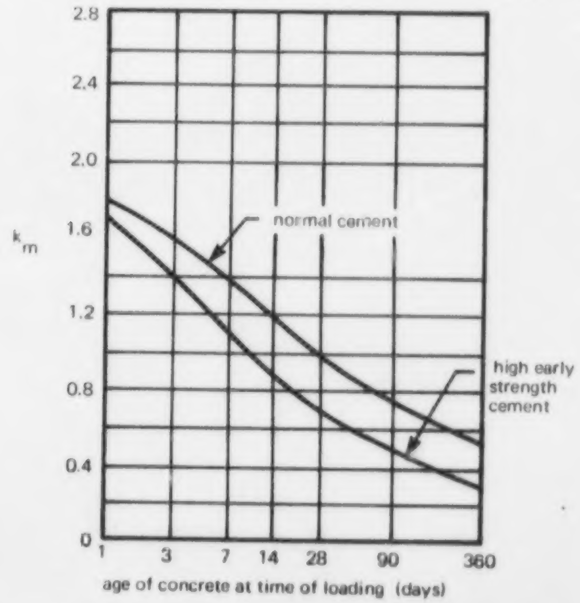
2—4.5.1 **General**

2—4.5.1.1 Clause 2—4.5 provides design wind loads for bridge structures, luminaire, sign and traffic signal supports, barriers and structural elements. All wind loads based on the reference wind pressure shall be treated as equivalent static loads. The effects of vortex shedding shall be determined from a dynamic analysis.

2—4.5.1.2 **Reference Wind Pressure**

The reference wind pressure, q, shall not be less than that given in Table A2—1.7 of Appendix A2—1 for a return period of:

Figure **Coefficient k_m** **2—4.4.5**



- 50 years for bridge structures, luminaire support structures exceeding 16 m in height, and overhead sign structures.
- 25 years for luminaire, traffic signal support structures 16 m or less in height, and barriers.
- 10 years for roadside sign structures where a long life expectancy is not required.

If the topography at the structure site can cause funnelling of the wind, the reference wind pressure shall be increased by 20%.

2 — 4.5.1.3 Gust Effect Coefficient

For structures that are not sensitive to wind action, which includes highway bridges with spans not exceeding 125 m, the gust effect coefficient, C_g, shall be taken as 2.0.

For slender, lighter structures, such as pedestrian bridges, luminaire, sign and traffic signal supports, barriers, and slender structural elements, C_g shall be taken as 2.5.

For structures which are sensitive to wind action, C_g shall be determined on the basis of a detailed analysis of dynamic wind action, using an approved method.

2 — 4.5.1.4 Exposure Coefficient

The exposure coefficient, C_e, shall not be less than 1.0 and shall be taken from Table 2 — 4.5.1.4 or computed from:

$$C_e = (0.10\,H)^{0.2}$$

Table 2 — 4.5.1.4 Wind Exposure Coefficient C_e

Height H, m		Exposure Coefficient C_e
	0 to 10	1.0
over	10 to 16	1.1
over	16 to 25	1.2
over	25 to 37	1.3
over	37 to 54	1.4
over	54 to 76	1.5
over	76 to 105	1.6

where H is the height above ground of the top of the superstructure. For luminaire, sign and traffic signal supports, and barriers, H shall be taken to the top of the standard, support, or structure considered. The height above ground shall be measured from the foot of cliffs, hills, or escarpments when the structure is located in uneven terrain, or from the low water level for structures over water.

2 — 4.5.1.5 Non-Uniform Loading

Wind loads shall be applied uniformly or non-uniformly over the entire structure, whichever produces the more critical effects. Unless specifically provided for, the non-uniform loading shall comprise 0.75 times the effective uniformly-distributed load over any portion of the area and the full effective uniformly-distributed load applied over the remaining area.

2 — 4.5.1.6 Overturning and Overall Stability

In addition to applying the prescribed loads in the design of members, overturning, uplift, and lateral displacement shall be considered.

2 — 4.5.1.7 Alternative Methods

When approved, representative wind tunnel tests or more detailed methods of analysis may be used to establish load coefficients or design criteria different from those specified.

2 — 4.5.2 Design of Superstructure

2 — 4.5.2.1 General

The superstructure shall be designed for wind induced vertical and horizontal drag loads acting simultaneously. The assumed wind diraction shall be perpendicular to the longitudinal axis for a straight structure or to an axis chosen to maximize wind-induced effects for a structure curved in plan.

2 — 4.5.2.2 Horizontal Drag Load

The following wind load per unit exposed frontal area of the superstructure shall be applied horizontally:

$$F_h = q\,C_e\,C_g\,C_h$$

where q, C_e and C_g are as specified in Clause 2 — 4.5.1 and C_h = 2.0.

In the case of truss spans, this load shall be taken to act on both windward and leeward trusses simultaneously unless a recognized method is used to calculate the shielding effect of the windward truss.

2—4.5.2.3 Vertical Load

The following wind load per unit exposed plan area shall be applied vertically:

$$F_v = q\,C_e\,C_g\,C_v$$

where q, C_e and C_g are as specified in Clause 2—4.5.1 and $C_v = 1.0$.

The vertical load F_v shall be taken to act either upwards or downwards.

In addition to the application of F_v as a uniformly-distributed load over the whole plan area, the effect of possible eccentricity in the application of the load shall be considered. For this purpose, the same total load shall be applied as an equivalent vertical line load at the windward quarter point of the transverse superstructure width.

2—4.5.2.4 Wind Load on Live Load

The horizontal wind load per unit exposed frontal area of the live load shall be computed using the expression F_h given in Clause 2—4.5.2.2 except that C_h shall be taken as 1.2. The exposed frontal area of the live load shall be the entire length of the superstructure, as seen in elevation in the direction of the wind as defined in Clause 2—4.5.2.1, or any part or parts of that length producing critical response, multiplied by a height of 3.0 m above the roadway surface for vehicular bridges and 1.5 m for pedestrian bridges. Areas below the top of a solid barrier wall shall be neglected.

2—4.5.3 Design of Substructure

2—4.5.3.1 General

The substructure shall be designed for wind-induced loads transmitted to it from the superstructure and wind loads acting directly on the substructure. Loads for wind directions both normal to and skewed to the longitudinal centreline of the superstructure shall be considered.

2—4.5.3.2 Wind Loads Transmitted from the Superstructure

The horizontal drag load specified in Clause 2—4.5.2.2 shall be resolved into transverse and longitudinal components using the skew angle modification coefficients given in Table 2—4.5.3.2. These loads shall be applied as equivalent horizontal line loads at the elevation of the centroid of the exposed frontal area of the superstructure.

The vertical load specified in Clause 2—4.5.2.3, modified for skew angle using appropriate coefficients from Table 2—4.5.3.2, shall be applied as an upward or downward line load either along the centreline of the superstructure or along the windward quarter point, whichever produces the more critical effect.

The vertical load and the longitudinal and transverse horizontal loads shall be applied simultaneously and the combination leading to maximum load effects in the substructure shall be used.

The provisions of Clause 2—4.5.2.4 shall apply in determining the wind load on the live load which is to be transferred to the substructure.

The modifications specified for "Other Spans" in Table 2—4.5.3.2 shall apply to skewed wind loads on the live load on any type of span.

2—4.5.3.3 Loads Applied Directly to Substructure

The substructure shall be designed for directly-applied horizontal drag loads. The wind load on a unit frontal exposed area of the substructure shall be computed using the expression for F_h given in Clause 2—4.5.2.2. The horizontal drag coefficient, C_h, shall be taken as 0.7 for circular piers, 1.4 for octagonal piers, and 2.0 for rectangular and square piers.

For wind directions skewed to the substructure, the loads shall be resolved into components taken to act perpendicular to the end and side elevations of the substruc-

Table 2—4.5.3.2 Modification of Wind Loads on Superstructure with Skew Angle

Skew Angle Measured from a Line Normal to Longitudinal Axis	Modification Coefficients			
	Truss Spans		Other Spans	
	Transverse Horizontal or Vertical Load	Longitudinal Horizontal Load	Transverse Horizontal or Vertical Load	Longitudinal Horizontal Load
0°	1.00	0.00	1.00	0.00
15°	0.93	0.16	0.88	0.12
30°	0.87	0.37	0.82	0.24
45°	0.63	0.55	0.66	0.32
60°	0.33	0.67	0.34	0.38

ture. These load components shall be assumed to act horizontally at the centroids of the exposed areas of the end and side elevations and shall be applied simultaneously with the loads transmitted from the superstructure.

2 — 4.5.4 Luminaire, Sign and Traffic Signal Supports, Barriers, and Slender Structural Elements

2 — 4.5.4.1 General

These structures shall be designed for horizontal drag loads at serviceability limit state Type II and ultimate limit states and where appropriate shall be designed for

Table **Horizontal Drag Coefficients** **2 — 4.5.4.2**

Type of Member	Shape of Members					
Vertical & Horizontal Structural Supports	Cylindrical	Dodecagonal*	Octagonal**	Flat***	Elliptical Broad Side Facing Wind	 Narrow Side Facing Wind
Single Member or Truss $D\sqrt{qC_e} \le 3.6$	1.2	1.2	1.2	1.7		
$3.6 < D\sqrt{qC_e} < 7.2$	$\dfrac{6.42}{(D\sqrt{qC_e})^{1.3}}$	$\dfrac{2.60}{(D\sqrt{qC_e})^{0.6}}$	1.2	1.7	$1.7\left(\dfrac{D_1}{D_2}-1\right) + C_{D1}\left(2-\dfrac{D_1}{D_2}\right)$	$C_{D2}\left[\dfrac{4-\dfrac{D_1}{D_2}}{3}\right]$
$D\sqrt{qC_e} \ge 7.2$	0.50	0.80	1.2	1.7		
Two members or trusses, one in front of the other	1.9			2.9		
Three trusses forming triangular cross-section	1.7			2.6		
Traffic Signals					1.2	
Luminaires with rounded surfaces					0.5	
Luminaires with rectangular flat surfaces					1.2	
Sign Panels and Barriers		Ratio of Sides = 1		Ratio of Sides > 1 ≤ 10	Ratio of Sides > 10	
		1.1		1.2	1.3	

D	Width or diameter of member, m.
D_1/D_2	Ratio of major to minor diameter of ellipse, maximum value of 2.
C_{d1}	Drag coefficient of cylindrical shape with diameter D_1.
C_{d2}	Drag coefficient of cylindrical shape with diameter D_2.
*	Valid for member having a ratio of corner radius to distance between parallel faces equal to or greater than 0.125.
**	The corners are assumed to be slightly rounded for octagonal sections; with sharp corners, a coefficient of 1.4 should be used.
***	Flat members are those shapes which are essentially flat in elevation, including plates, angles and squares with slightly rounded corners. A coefficient of 2.0 shall be used for single flat members, including plates, angles and squares with sharp corners.

lateral or across-wind load effects induced by vortex shedding at serviceability limit state Type I.

Details which may cause stress concentration at critical sections shall be considered in the design or, preferably, avoided.

2 — 4.5.4.2 Horizontal Drag Load

The wind-induced horizontal drag load acting per unit exposed frontal area of structural elements, signs, barriers, luminaires and traffic signals shall be as specified in Clause 2 — 4.5.2.2 using values of C_h as prescribed in Table 2 — 4.5.4.2.

2 — 4.5.4.3 Assessment of Wind Loads Acting on Various Components

The horizontal drag load on components of signs, luminaire and traffic signal support structures shall be in accordance with Clause 2 — 4.5.4.2.

When the effects of wind loads are to be combined with those of ice accretion loads, the increase in exposed area caused by ice accretion shall be considered.

For luminaires, traffic signals, and regulatory traffic signs which are less than 1 m² in area, the exposed frontal area for all wind directions shall be taken as constant and equal to the maximum frontal exposed area in any direction.

The action of wind loads shall be as shown for typical structures in Figure 2 — 4.5.4.3 and in accordance with the following

a) Horizontal supports of luminaire support structures shall be designed for wind loads W_l and W_h applied normal to the supports and at the centroids of the respective areas

b) Horizontal supports of sign bridges, butterfly and cantilever structures and traffic signal supports shall be designed for wind loads W_h and W_p applied normal to the supports and at the centroids of the respective areas.

c) The vertical supports for sign and traffic signal structures, where the signs are nominally parallel, shall be designed for the effects of wind from any direction by applying normal and transverse wind loads acting simultaneously in the combinations given in Table 2 — 4.5.4.3

Table 2 — 4.5.4.3 Normal and Transverse Load Combinations

Case	Normal Component	Transverse Component t_c
1	1.0 BL*	0.2 BL
2	0.6 BL	0.3 BL

*BL = Basic Load

The Basic Load referred to in Table 2 — 4.5.4.3 shall be the effects from the wind loads, W_v, W_p and W_h applied at the centroids of the respective areas and normal to the sign faces. The transverse components shall be equally distributed to all supports. Roadside signs with two or more vertical supports shall be designed for normal wind loads only.

d) For vertical supports carrying two or more arms, the maximum torque shall be computed assuming that wind loads may act on any one or more arms. The resulting torsional effects shall be included with other effects due to full wind load.

For high-mast lighting structures, post-type luminaire supports and roadside signs with single supports, the torque shall be not less than the total wind load multiplied by 0.15 times the overall width of the luminaire cluster or sign.

2 — 4.5.4.4 Across-Wind Loads

Cylindrical and prismatic luminaire, sign and traffic signal supports, and other slender exposed structural elements, shall be designed to resist the dynamic effects of lateral or across-wind loads induced by vortex shedding at serviceability limit state Type I.

The excitation due to vortex shedding shall be treated as a sinusoidally varying load with a frequency n_e calculated as:

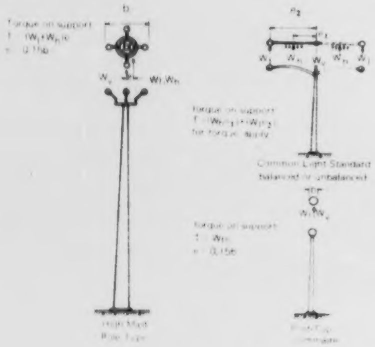
$$n_e = S\frac{V}{D}$$

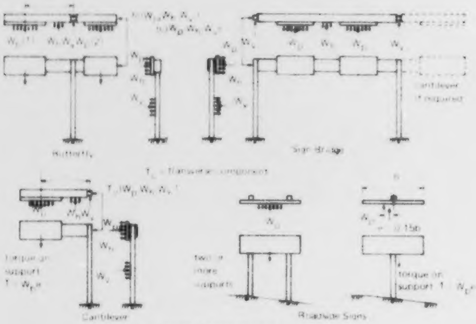
where: V = the mean wind speed at the top of the structure or at the level of the element considered;

D = the diameter of a circular member or the frontal width of a prismatic member or structural shape;

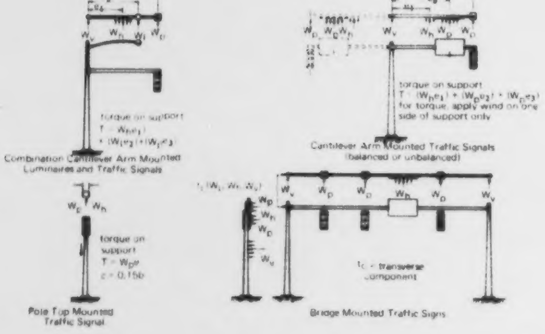
S = the Strouhal Number of a particular cross-sectional geometry specified in Table A2 — 2.2.

Figure **Loads on Sign, Luminaire, and Traffic Signal Support Structures** **2 — 4.5.4.3**


b
Torque on support
T = (W_l+W_h)e
e = 0.15b
W_v
W_l,W_h
High Mast
Pole Type
e_2
e_1
W_l
W_h
W_v
W_h
W_l
Torque on support
T = (W_h e_1)+(W_l e_2)
for torque, apply
Common Light Standard
balanced or unbalanced
W_l,W_v
Torque on support
T = W_h e
e = 0.15b
Post Top


W_p(1)
W_h W_v W_p(2)
W_p
W_h
W_v
W_p
W_v
W_p
W_h
W_p
W_v
W_h
W_v
Cantilever
if required
Butterfly
Sign Bridge
T_c = transverse component
T_c(W_p,W_h,W_v)
W_p
W_h W_v
W_p
W_h
W_v
torque on
support
T = W_p e
Cantilever
W_p
two or
more
supports
b
W_p
e = 0.15b
torque on
support T = W_p e
Roadside Signs

Loads on Sign Support Structure


e_3
e_2
e_1
W_v
W_h
W_l
W_p
torque on support
T = W_h e_1
+ (W_l e_2)+(W_p e_3)
Combination Cantilever Arm Mounted
Luminaires and Traffic Signals
e_3
e_2
e_1
W_p
W_p
W_h
W_v
W_h
W_p
W_p
torque on support
T = (W_h e_1) + (W_p e_2) + (W_p e_3)
for torque, apply wind on one
side of support only
Cantilever Arm Mounted Traffic Signals
(balanced or unbalanced)
W_p
W_h
torque on
support
T = W_p e
e = 0.15b
Pole Top Mounted
Traffic Signal
T_c(W_p,W_h,W_v)
W_v
W_p
W_p
W_h
W_p
W_v
W_p
W_h
W_p
W_v
T_c = transverse
component
Bridge Mounted Traffic Signs

Loads on Traffic Signal Support Structure

Resonant vibrations shall be assumed to occur when the frequency of vortex shedding coincides with a natural frequency of the structure. The significance of vortex shedding effects for a particular structure or structural element shall be examined for all modes of vibration for which the natural frequency $n \leq 1.24(S/D)\sqrt{qC_e}$ where q is the reference wind pressure for the design return period.

Procedures used for estimating the dynamic effects of vortex shedding shall be as given in Appendix A2 — 2. The stress range limits specified in Section 11 for a fatigue life of over 2 000 000 cycles shall apply unless a detailed fatigue damage analysis shows that other limits are appropriate.

Ice Accretion Loads on Exposed Surfaces **2 — 4.6**

Ice accretion loads shall be taken to occur on all exposed surfaces of superstructure members, structural supports, traffic signals, luminaires and railings. In the case of sign panels, bridge girders and solid barriers, ice accretion shall be considered to occur on one side only. The design ice thickness shall be the value given in Figure A2 — 1.4 of Appendix A2 — 1. A unit weight of 9.8 kN/m³ shall be used in computing ice accretion loads.

Loads Due to Ice Pressure **2 — 4.7**

2 — 4.7.1 **General**

Ice forces on piers shall be estimated with regard for site conditions and the types of ice action expected. Consideration shall be given to:

- dynamic ice pressure due to moving ice sheets and ice floes carried by stream flow, wind, or currents, in accordance with Clause 2 — 4.7.2;
- static ice pressure due to thermal movements of continuous stationary ice sheets on large bodies of water, in accordance with Clause 2 — 4.7.3;
- static pressure resulting from ice jams;
- static uplift or vertical loads resulting from adhering ice in waters of fluctuating level.

The expected thickness of ice, the direction of its movement, and the height at which it acts shall be determined by field investigations, published records, aerial photography or other suitable means. Consideration shall be given to the worst expected combinations of height, thickness and pressure, to the possibility of unusual thicknesses resulting from special circumstances or operations and to the natural variability of ice conditions from year to year.

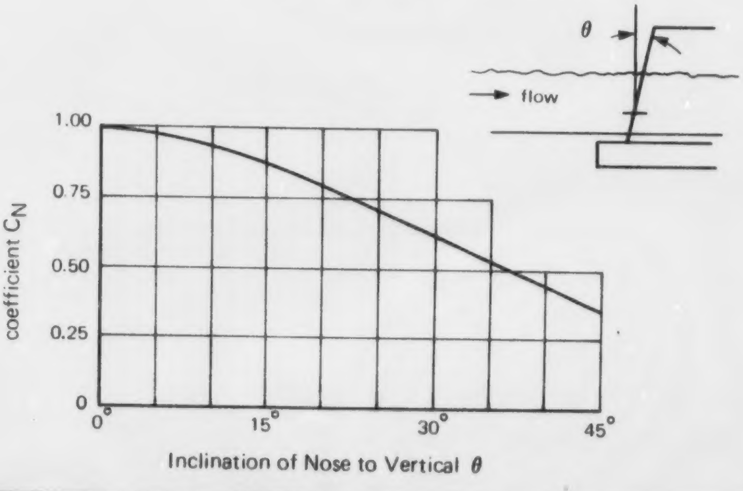
2 — 4.7.2 **Dynamic Ice Pressure**

The horizontal load resulting from the pressure of moving ice shall be calculated from:

$$F = C_N C_A ptw$$

where:
p = ice pressure as specified below;
C_N = coefficient of nose inclination from Figure 2-4.7.2;
$C_A = \sqrt{5\frac{t}{w} + 1}$.

Figure **Coefficient of Nose Inclination** **2 — 4.7.2**



When the inclination of the nose to the vertical, θ, is 30° or more, the width of the pier need not be taken as more than $2t$, provided that the nose is semi-circular, or wedge shaped.

The value of ice pressure, p, shall be selected from (a) through (d) of this clause, and shall be based on an assessment of the probable condition of the ice at the time of movement, on previous local experience, and on an assessment of the performance of existing structures:

a) $p = 375$ kPa where break-up occurs at melting temperatures and the ice is substantially disintegrated in its structure and runs as small "cakes", i.e., flat pieces of an ice sheet which are smaller in plan area than a horizontal section through the pier shaft at the ice level;
b) $p = 750$ kPa where break-up occurs at melting temperatures, but the ice moves in large pieces and is internally sound;
c) $p = 1125$ kPa where, at break-up, there is an initial movement of the ice sheet as a whole or where large sheets of sound ice may strike the piers;
d) $p = 1500$ kPa where break-up or major ice movement may occur, with ice temperature significantly below the melting point.

Piers shall be placed with longitudinal axes parallel to the principal direction of ice action. The horizontal load, F, shall be taken to act along the direction of the longitudinal axis. A load transverse to the longitudinal axis of not less than $0.15F$ shall be considered to act simultaneously. Where the longitudinal axis of a pier cannot be placed parallel to the principal direction of ice motion, or where the direction of ice motion may shift, the load F shall be calculated using a frontal width, w, appropriate to the direction of ice motion, and shall be resolved into transverse and longitudinal vector components. In such conditions, the transverse load shall be taken as not less than 20% of the total load.

The ice loads specified shall apply only to piers of substantial mass and dimensions. Solid shaft piers having a thickness of at least 1 m, a height from the top of the footing to the centre of ice action of not more than three times the shaft thickness, and a length of at least four times the thickness, may in general be considered to have substantial mass and dimensions. In other cases, the ice loads shall be subject to approval.

Slender and flexible piers, such as circular piers, caissons, or piles having a total height greater than seven times the diameter shall not be used where significant dynamic ice loads can occur.

2—4.7.3 Static Ice Pressure

Ice pressure due to thermal movements, ice jams, uplift or vertical loads resulting from ice adhering to piers shall receive special consideration.

Earthquake Effects 2—4.8

2—4.8.1 General

Bridges shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure, in accordance with Clauses 2—4.8.2 to 2—4.8.7.

2—4.8.2 Equivalent Static Load Method

For structures with supports or supporting members of approximately equal horizontal stiffness, an equivalent horizontal static load, Q, shall be applied at the centre of mass of the structure. In the distribution of the load, the stiffness of the superstructure and supporting members, abutment restraint, and the deflected position of the structure shall be considered. The load Q shall be:

$$Q = CIFW$$

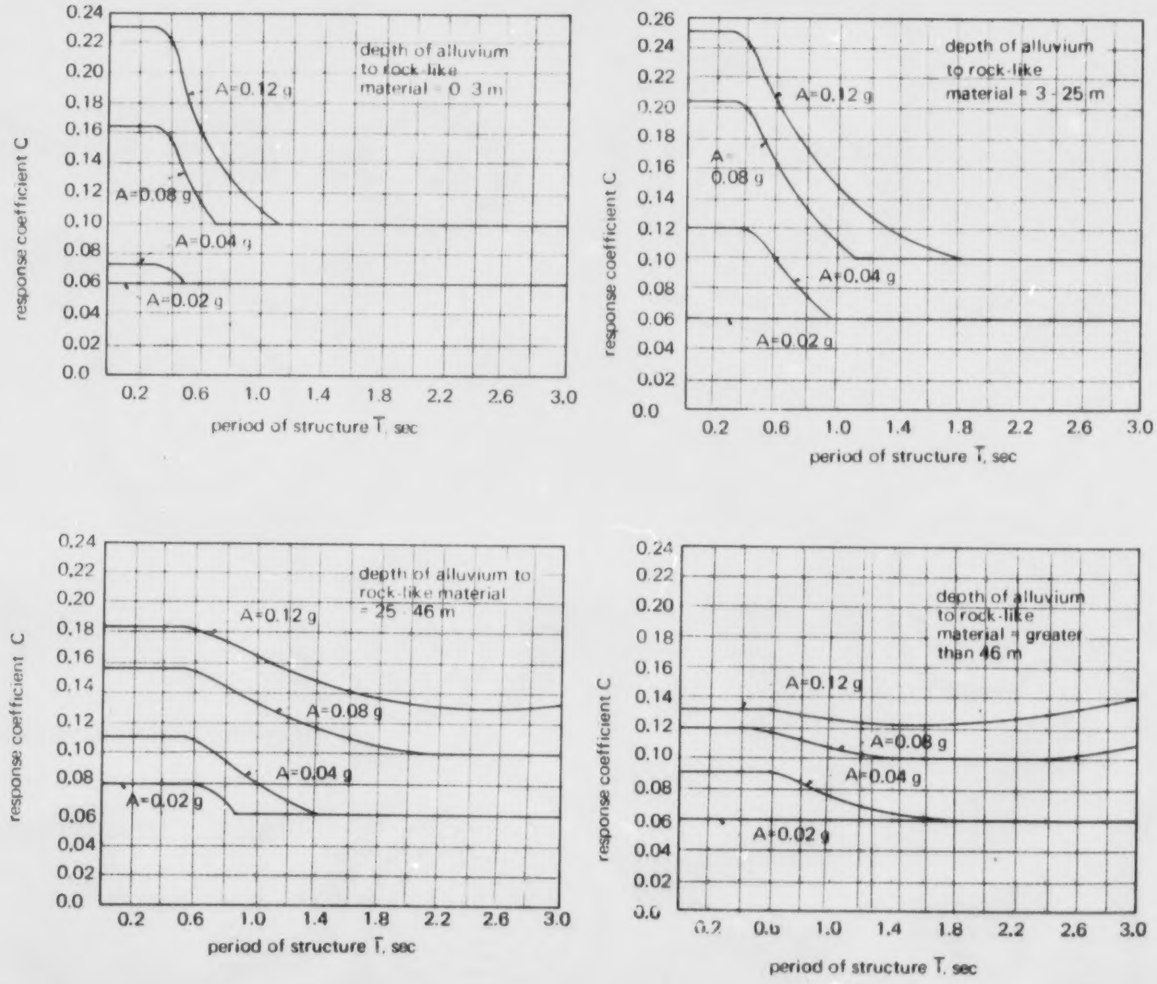
where:
$F = 1.0$ for structures where single columns or piers resist the horizontal loads;
$F = 0.8$ for structures where continuous frames resist horizontal loads applied along the frame;
$I = 1.3$ for all bridges designed for post-disaster service and 1.0 for all other bridges;
C shall be as given in Figures 2—4.8.2(a), (b), (c) and (d), except that for locations in Seismic Zone 0, the value of C shall be 0.01.

The acceleration ratios, A, shown in these figures are the acceleration ratios given in Appendix A2 — 1 which represent the minimum value for a site anywhere within a particular zone. The value T shall be computed for the mode of vibration producing critical response, using dynamic analysis techniques or, for usual types of structures, shall be taken as:

$$T = 0.32\sqrt{W/P}$$

where: P = the total uniformly-distributed force required to cause a 25 mm deflection of the structure in the direction of the load Q and at its point of application.

Figure **Response Coefficient C for Various Values of Acceleration Ratio A** **2 – 4.8.2**



2—4.8.3 **Response Spectrum Method**

For complex structures and for structures with an unusual distribution of mass or stiffness, a response spectrum dynamic approach shall be used for seismic analysis. The combined response curves for C given in Figures 2—4.8.2(a), (b), (c) and (d), modified by the framing coefficient, F, and the importance coefficient, I, shall be used as the design response spectrum in the absence of more suitable data.

2—4.8.4 **Special Cases**

Structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 3.0 s, shall be considered as special cases. These structures shall be designed using current seismicity, soil response and dynamic analysis techniques.

2—4.8.5 **Design of Restraining Features**

Restraining features to limit the displacement of the structure, e.g., hinge ties, shear blocks, etc., shall be designed for the load:

$$Q_R = 0.25\,W_c - \Sigma Q_c$$

where: ΣQ_c is the sum of the column shears due to Q,
and W_c shall be determined by examining the entire structure on each side of the restraining feature.

2—4.8.6 **Bearings and Supports**

Where there is no effective restraint against transverse and longitudinal bearing movement, bearings and supports shall be designed in such a manner that the structure can undergo, without collapse, movements equal to six times the movements determined by elastic analysis from the loads specified in Clauses 2—4.8.2, 2—4.8.3 or 2—4.8.4. The design of the structure shall be such as to prevent excessive rocking motions due to exceptionally unfavourable foundation conditions.

2—4.8.7 **Detailing**

The design of structures in Seismic Zones 2 and 3 shall comply with the detailing provisions for structural concrete given in the "Special Provisions for Seismic Designs", CSA Standard CAN-A23.3-M77, *Code for the Design of Concrete Structures for Buildings*.

Loads Due to Stream Current **2—4.9**

2—4.9.1 **Load Along Longitudinal Axis of Pier**

The load caused by flowing water, and acting in the longitudinal direction on piers and on drift which may lodge against them, shall be calculated from:

$$P = \frac{C_D A \gamma V^2}{2}$$

where C_D is the drag coefficient given in Table 2—4.9.1.

2—4.9.2 **Lateral Load**

The lateral, uniformly-distributed load on a pier shaft due to water flowing at an angle to the longitudinal axis shall be calculated from:

$$P_P = \frac{C_L H L \gamma V^2}{2}$$

where C_L is the lateral load coefficient given in Table 2—4.9.2.

Table **Drag Coefficient C_D** **2—4.9.1**

Type	C_D
Semi-circular nosed	0.7
Square ended	1.4
Drift lodged against pier	1.4
Wedge nosed, included angle 90° or less	0.8

Table **Lateral Load Coefficient C_L** **2—4.9.2**

Angle Between Direction of Flow and Longitudinal Pier Axis	C_L
0°	0.0
5°	0.5
10°	0.9
20°	0.9
30° and greater	1.0

Collision Load **2 — 4.10**

2 — 4.10.1 Highway bridge piers less than 10 m from the edge of the road pavement shall be designed for an equivalent horizontal static force of 1000 kN.

2 — 4.10.2 The collision load shall be applied 1.20 m above ground level at the pier, horizontally at 10° to the direction of travel.

Construction Loads and Loads on Temporary Structures **2 — 4.11**

2 — 4.11.1 General

The weights of materials, men, and equipment supported during construction shall be considered as dead loads or live loads in accordance with the definitions and with Clauses 2 — 4.11.2 and 2 — 4.11.3.

In the calculation of loads due to wind, ice, stream flow, and earthquake, return periods and the possibility of occurrence shall be determined in accordance with the expected life of the structure or the duration of the stage considered.

2 — 4.11.2 Dead Loads

Dead loads shall include weights of formwork, falsework, fixed appendages, stored material, and lifting and launching devices or parts thereof which are not subject to movement during the stage considered.

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2 — 4.11.3 Live Loads

Live loads shall include weights of men, vehicles, hoists, cranes, and structural components which are subject to movement during the stage considered.

2 — 4.11.4 Falsework

Falsework shall be designed and detailed entirely in accordance with CSA Standard S269.1-1975, *Falsework for Construction Purposes*.

Load Factors and Loading Combinations **2 — 5**

General **2 — 5.1** The loading combinations to be considered and the load factors to be used shall be as shown in Table 2 — 5.1(a) and (b) unless otherwise specified in Clause 2 — 5.

Total Factored Load Effect **2 — 5.2**

2 — 5.2.1 For each loading combination, every load that is to be included shall be multiplied by the load factor specified and the load effects shall be computed. The factored load effects shall then be added together to obtain the total factored load effect.

2 — 5.2.2 Each load effect shall include all significant effects due to elastic distortions caused by the load.

Permanent Loads **2 — 5.3**

2 — 5.3.1 In obtaining total factored load effects, all permanent loads shall be included except those which, for the structure or at the stage considered, are not applied.

2 — 5.3.2 For ultimate limit states loading combinations, the maximum or minimum values given in Table 2 — 5.1(b) shall be used for each load factor, to maximize each total factored load effect, except that for dead loads, provided inspection or supervision of construction ensures a close compliance with the plans, a factor of 1.00 shall be used instead of the minimum values.

Minimum values of the load factors shall not be used together with maximum values for other loads when the possibility of the loads having minimum and maximum values simultaneously can safely be excluded.

2 — 5.3.3 In the calculation of overturning moments for cantilever earth-retaining structures, when the maximum value of α_E for active pressure is used, the value of α_D shall be 1.25 for the retained soil.

Table **Load Combinations and Load Factors** **2 — 5.1(a)**

	Permanent Loads (For U.L. States, use max. or min. value of α as specified in Clause 2 — 5.3.2)			Transitory Loads					Exceptional Loads (Use Only One)			
	D	E	P	L	K	W	V	S	Q	F	A	H
Serviceability Limit States												
Type I	1.00	1.00	1.00	0.80†	0	1.00*	0	0	0	0	0	0
Type II	1.00	1.00	1.00	0.75	0.80	0.70*	0	1.00	0	0	0	0
Ultimate Limit States												
ULS Combination 1	α_D	α_E	α_P	1.40	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.25	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.15	1.00	0.40	0.40	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.30	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0.70*	0	0	1.30	1.30	1.30	1.40

* Include this load for structures covered by Clause 2 — 4.5.4 only.

† For Clause 2 — 6.2.2.2 use this load alone.

A	ice accretion load
D	dead load
E	loads due to earth presure and hydrostatic pressure other than dead load. Factors for surcharges shall be those of the loads causing them.
F	loads due to steam flow and ice pressure
H	collision load
K	limiting friction in sliding bearings, or the combined effect of all strains including those due to temperature change and differential, concrete shrinkage, differential shrinkage and creep, but excluding the effects of elastic distortion.
L	live load. All applicable loads from Clause 2 — 4.3, including dynamic load allowance when applicable.
P	secondary prestress effects
Q	earthquake load
S	load due to differential foundation settlement
V	wind load on L.L.
W	wind load on structure

Table **Maximum and Minimum Values of Load Factors** **2 — 5.1(b)**

Dead Load	Max α_D	Min α_D
Factory-produced components excluding wood	1.10	0.95
Cast-in-place concrete, wood and all non-structural components	1.20	0.90
Wearing surfaces, based on measured thickness.	1.20	0.90
Wearing surfaces, based on nominal or specified thickness.	1.50	0.65
Earth fill, negative skin friction on piles	1.25	0.80
Water	1.10	0.90
Earth Pressure & Hydrostatic Pressure	**Max α_E**	**Min α_E**
Passive earth pressure*	1.70	0.80
At-rest earth pressure	1.40	0.65
Active earth pressure	1.25	0.60
Hydrostatic pressure	1.10	0.90
Prestress	**Max α_E**	**Min α_E**
Secondary prestress effects	1.05	0.95

*When considered as a load. Passive earth pressure considered as a resistance is factored in accordance with Section 6.

2 — 5.3.4 In the calculation of overturning moments occurring during the cantilever construction of segmental concrete bridges, maximum and minimum values of α_D equal to 1.05 and 1.0 shall be assumed for the erected segments provided that construction controls are specified which ensure that the difference in weight between any two segments forming a balancing pair does not exceed 5% and that all differences in weight are corrected before the addition of further segments.

Transitory Loads **2 — 5.4**

2 — 5.4.1 Transitory loads shall be included in the loading combinations only if there is a possibility of the loads being applied to the structure at the stage considered and if their inclusion increases the total factored load effect.

2 — 5.4.2 For evaluation of a structure for the passage of a controlled vehicle, the live load due to the controlled vehicle shall be multiplied by a load factor of 1.15 in lieu of the values shown in Table 2 — 5.1(a). The modification factors of Clause 2 — 4.3.1.9 shall not apply to the load due to the controlled vehicle itself but the design lane containing the controlled vehicle shall be included in determining the number of loaded design lanes.

2 — 5.4.3 In the evaluation of structures to determine posted load restrictions, the load factor for live load may be taken as 1.25 provided the structure is to be re-evaluated within five years.

Exceptional Loads **2 — 5.5** From among the exceptional loads for which there is a possibility of occurrence for the structure and at the stage considered, the one load which produces the largest total factored load effect shall be included. Exceptional loads shall not be combined with transitory loads except that a combination of wind load and ice accretion loads shall be considered for structures covered by Clause 2 — 4.5.4.

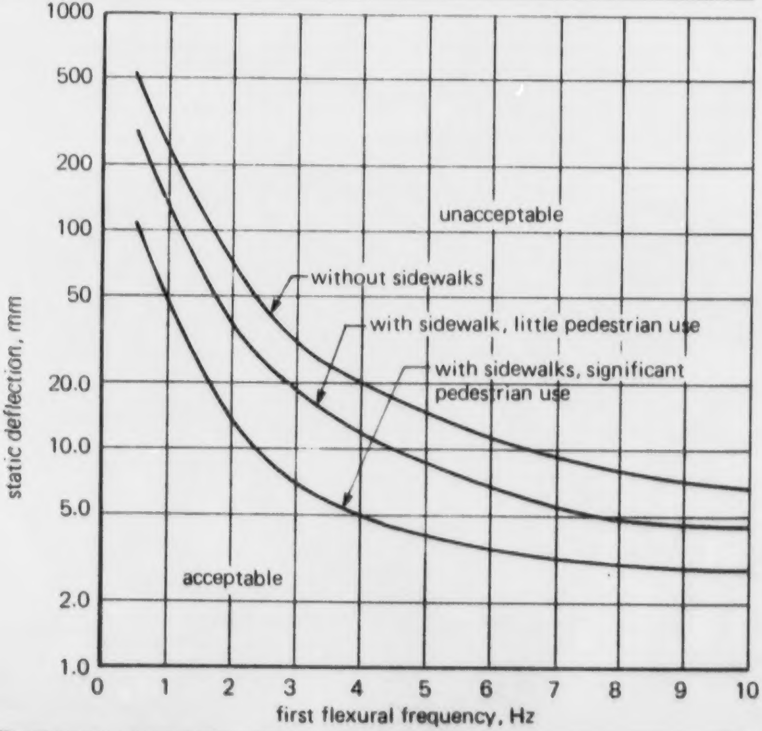
Limit States Criteria **2 — 6**

Ultimate Limit States **2 — 6.1**

2 — 6.1.1 No total factored load effect shall exceed the factored resistance provided to resist it.

2 — 6.1.2 In applying Clause 2 — 6.1.1 to Evaluation Levels 1, 2 and 3 and to construction loads, unless otherwise approved, the total factored load effect used for each

Figure **Deflection Limits for Serviceability** **2 — 6.2.2.2**



applicable loading combination shall not be less than 1.25 times the sum of the unfactored load effects which are to be included. Loads for which a load factor of zero is specified shall not be included.

2 — 6.1.3 Structures subject to overturning or uplift shall either be proportioned so that the total factored load effect does not result in overturning or uplift for any loading combination, or shall be provided with anchorages complying with Clause 2 — 6.1.1.

Serviceability Limit States **2 — 6.2**

2 — 6.2.1 **General**

Structural components shall satisfy the requirements for the serviceability limit states as specified in the applicable sections of the Code for the appropriate loading combination.

2 — 6.2.2 **Superstructure Vibration**

2 — 6.2.2.1 **General**

Vibration limitations, whether or not prescribed in the form of deflection limits, shall be serviceability limit state Type I requirements.

2 — 6.2.2.2 **Highway Bridges**

Superstructures shall be proportioned so that the maximum deflection due to the factored highway live load, including the dynamic load allowance, does not exceed the limit given in Figure 2 — 6.2.2.2 for the anticipated degree of pedestrian use. The deflection limit shall apply at the centre of the sidewalk or, if there are no sidewalks, at the face of the barrier wall or railing. The truck shall be positioned as specified in Clause 2 — 4.3.1.7.

2 — 6.2.2.3 **Pedestrian Bridges**

Superstructures shall be proportioned so that a single pedestrian weighing 700 N does not produce an acceleration exceeding the limit given in Figure 2 — 6.2.2.3 as a function of the first flexural frequency of the superstructure.

It shall be assumed that the pedestrian walks with a footfall frequency, f_f, that is the lesser of the first flexural frequency of the structure or 4 Hz. The stride assumed shall be 0.9 m or that which taken with the assumed footfall frequency results in a speed of 2.5 m/s, whichever is the lesser. The footfall force shall be taken as 180 sin $(2\pi f_f t)$, N.

Pedestrian bridges shall be designed so that the longitudinal and the lateral frequency of vibration of the superstructure shall not be less than 1.5 times the first flexural frequency of the superstructure, nor less than 4 Hz, whichever is the smaller.

Static values of the material properties shall be used for the dynamic analysis. An approved alternative calculation procedure may be used.

Figure **Acceleration Limit for Pedestrian Bridge Serviceability** **2 — 6.2.2.3**

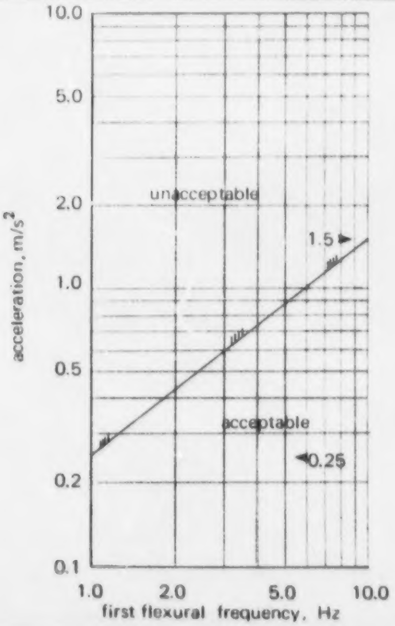
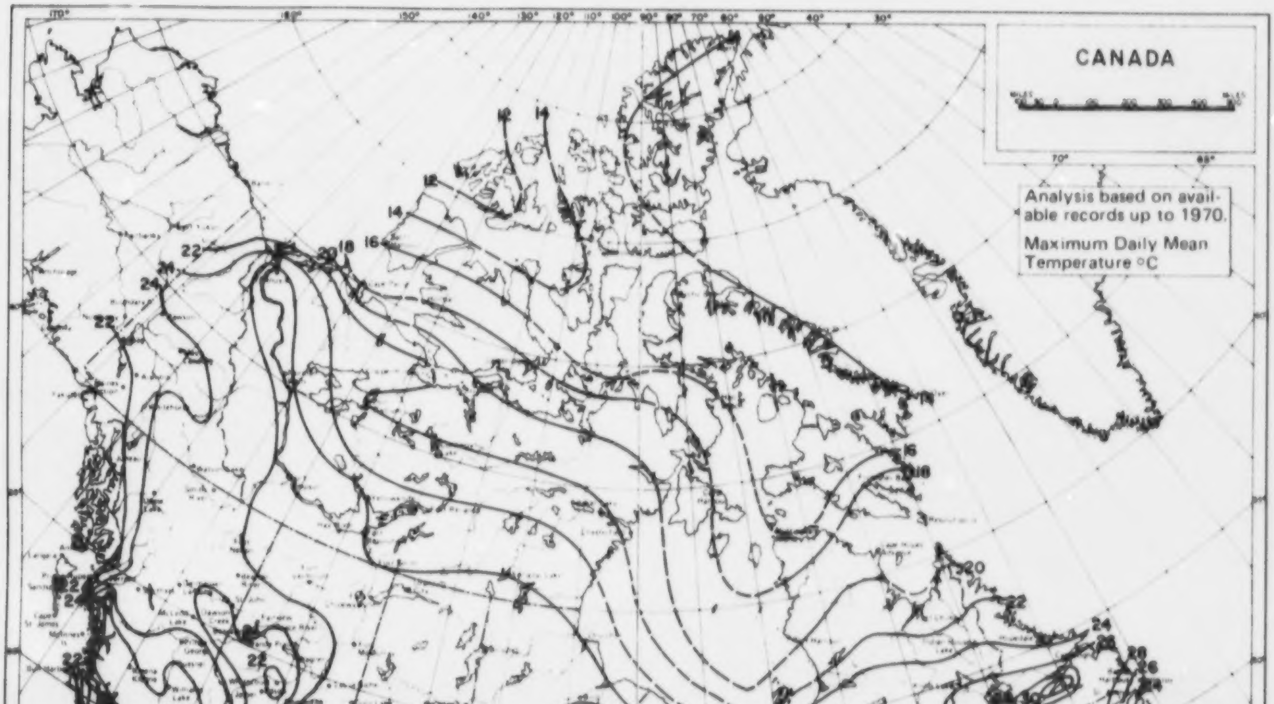


Figure **Maximum Daily Mean Temperature** **A2 — 1.1**

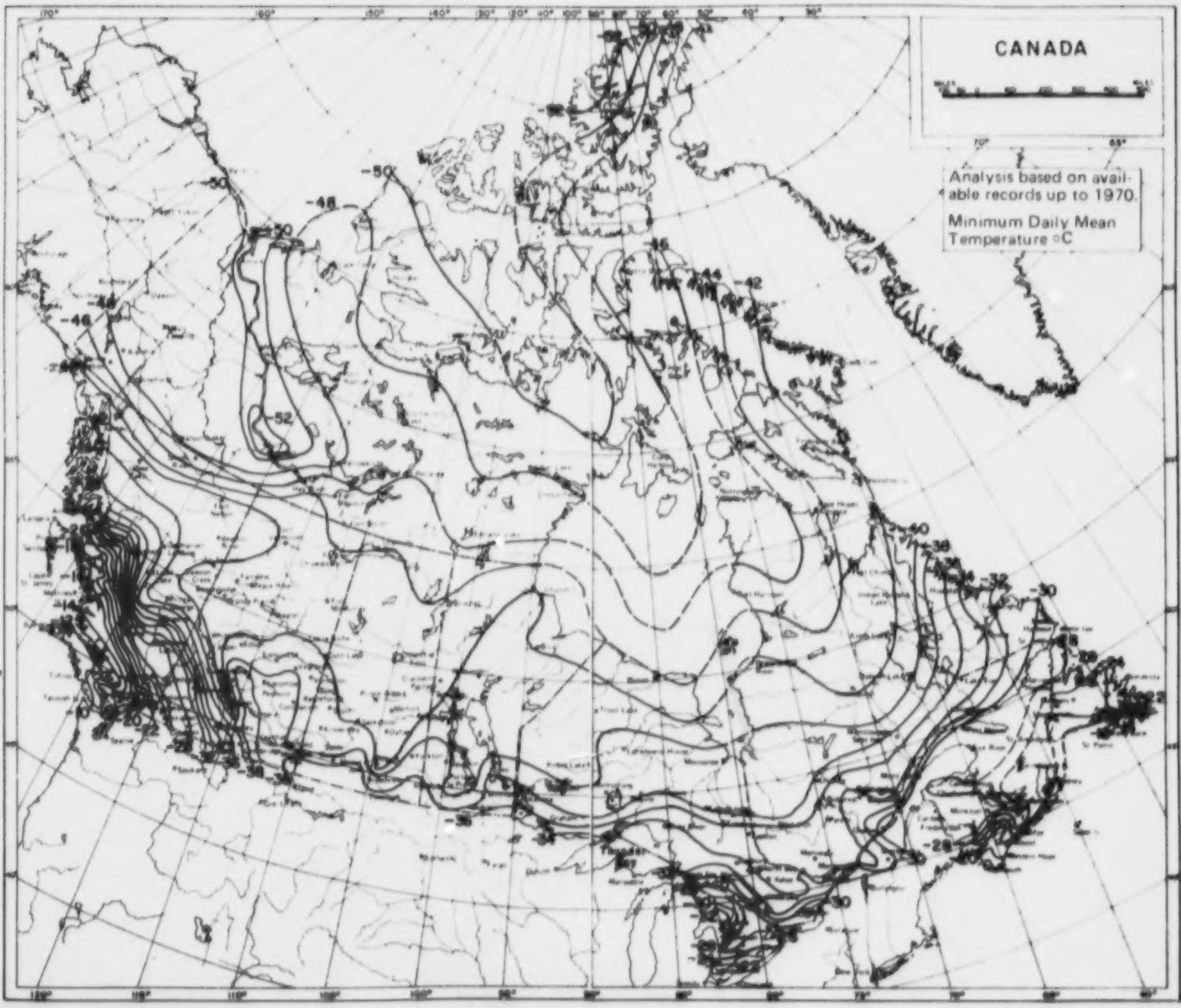

CANADA
MILES
70°
60°
120°
110°
100°
90°
80°
70°
60°
50°

Analysis based on available records up to 1970.

Maximum Daily Mean Temperature °C

modified equal area projection

Figure **Minimum Daily Mean Temperature** **A2 — 1.2**



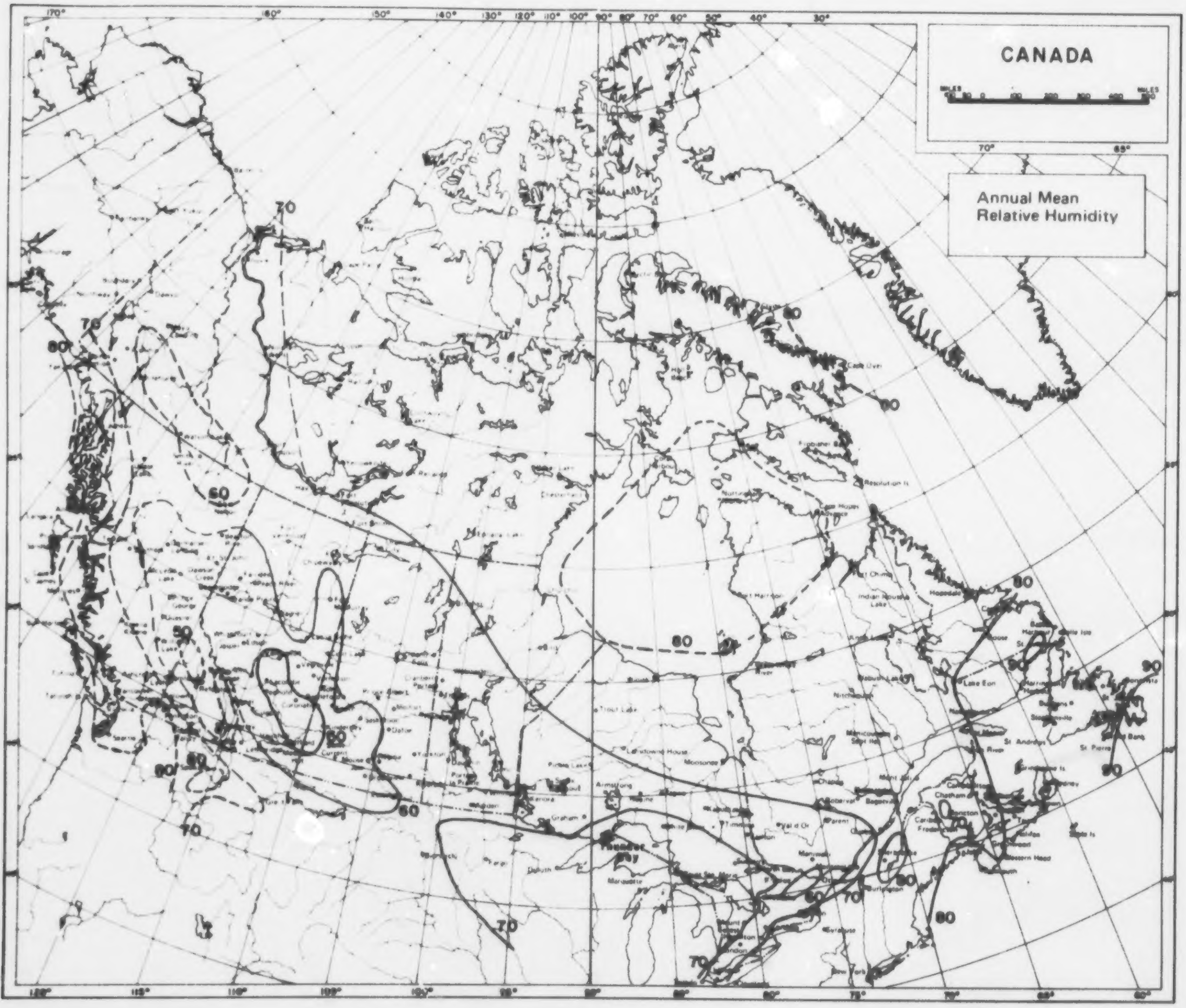
CANADA
-50
-48
-46
-52
-42
-40
-38
-36
-32
-30
-28
-22
70°
65°
120°
110°
100°
90°
80°
70°
60°

Analysis based on available records up to 1970.

Minimum Daily Mean Temperature °C

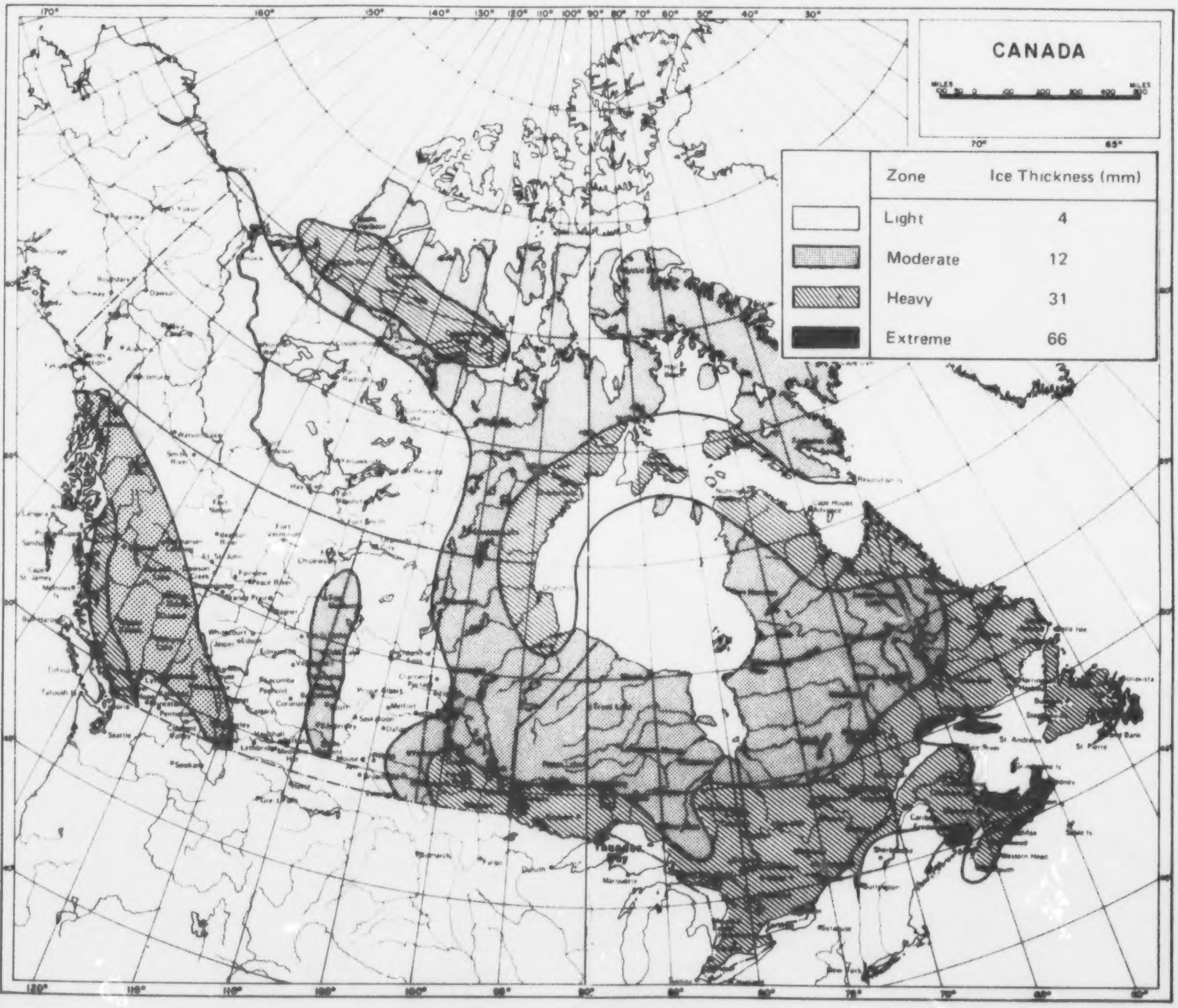
modified equal area projection

Figure **Annual Mean Relative Humidity** **A2 — 1.3**


CANADA
MILES
Annual Mean
Relative Humidity
70
70
80
60
50
60
60
80
70
60
80
80
80
60
80
90
90
90
90
80
70
70
80
70
80
70

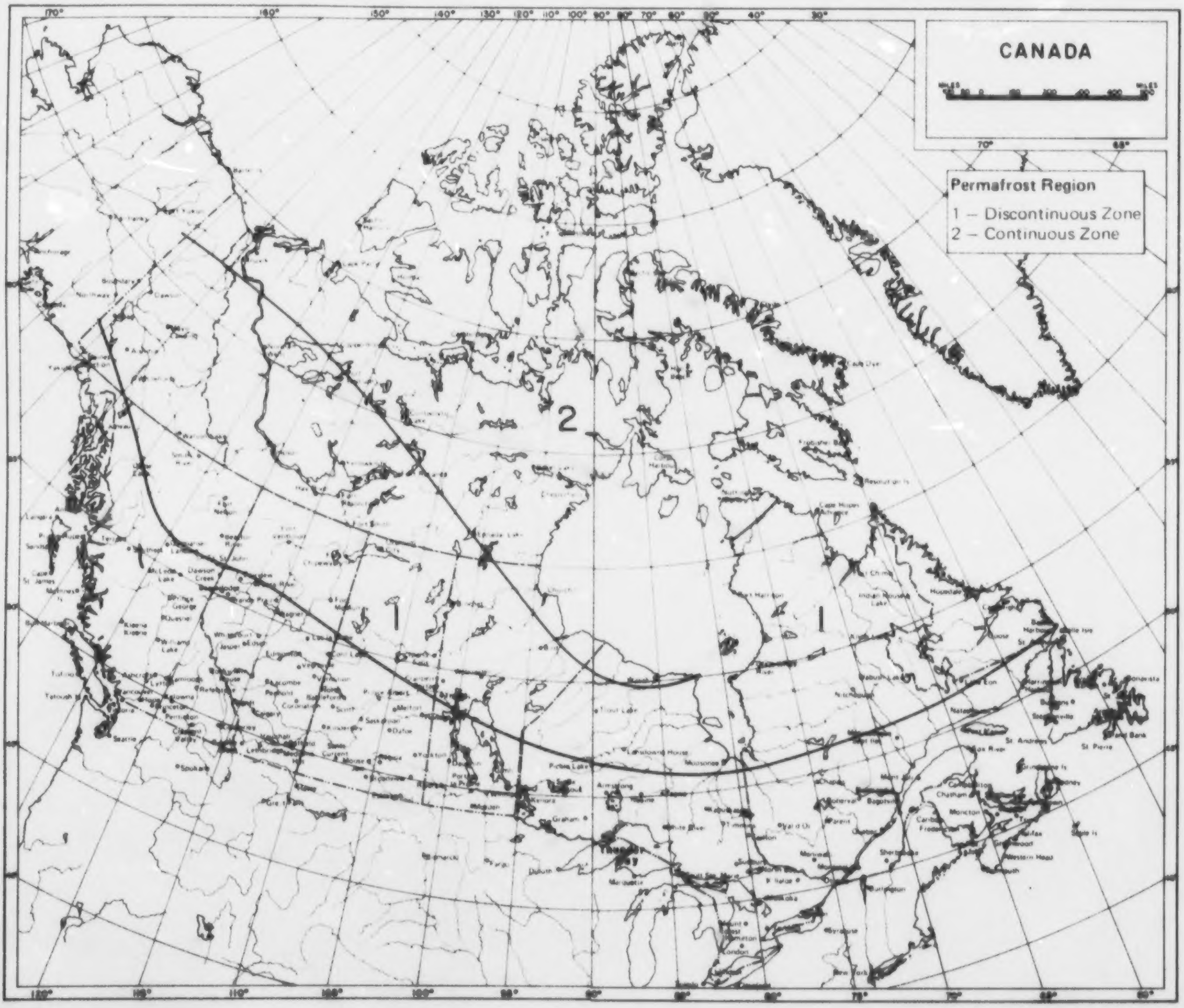
modified equal area projection

Figure **Ice Accretion** **A2 — 1.4**


CANADA
MILES
Zone
Ice Thickness (mm)
Light 4
Moderate 12
Heavy 31
Extreme 66

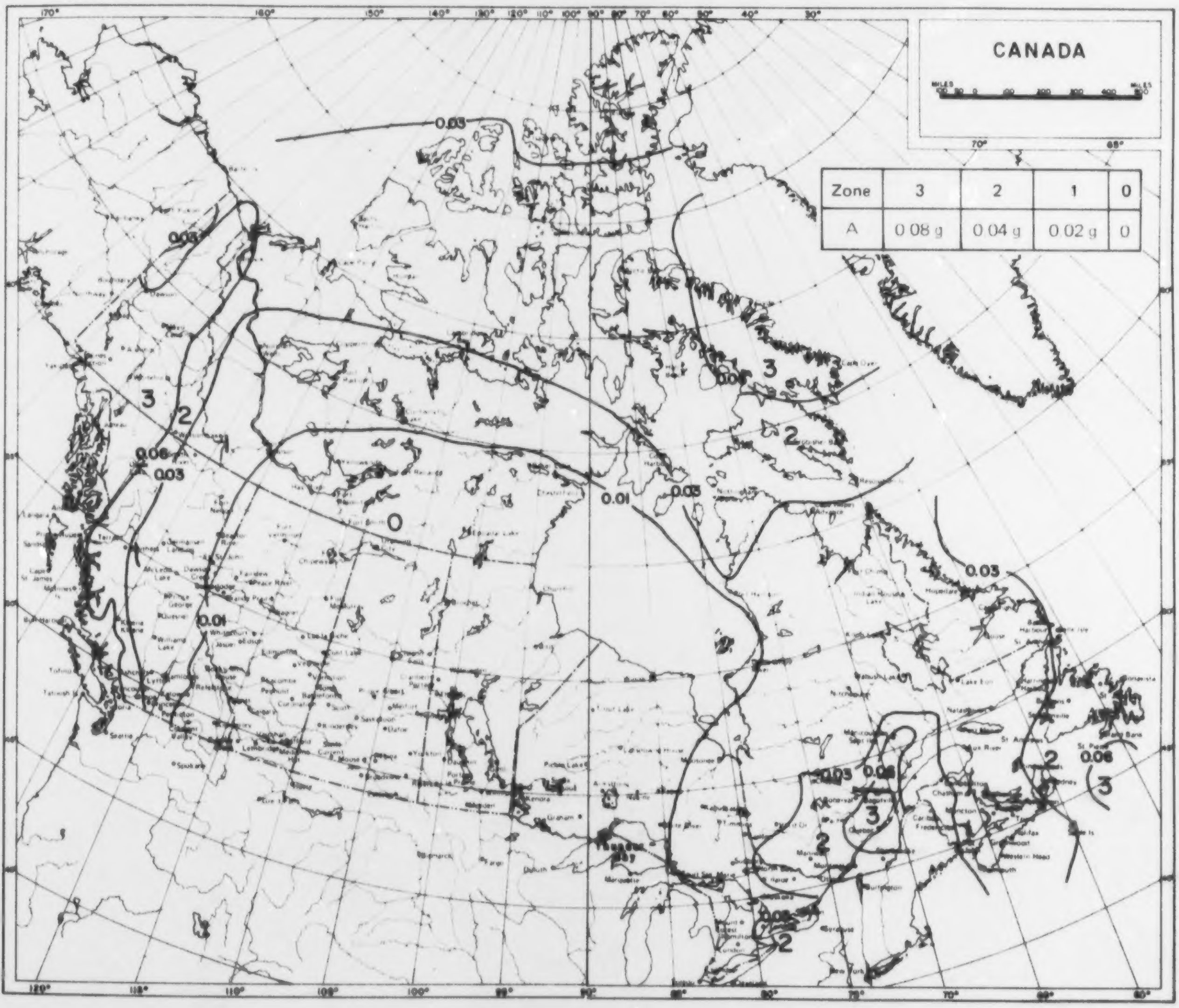
modified equal area projection

Figure **Permafrost Region** **A2 — 1.5**


CANADA
Permafrost Region
1 – Discontinuous Zone
2 – Continuous Zone
1
2
1
170°
160°
150°
140°
130°
120°
110°
100°
90°
80°
70°
60°
50°
40°
30°
120°
110°
100°
90°
80°
75°
70°
65°
60°

modified equal area projection

Figure **Seismic Zones with Boundary Acceleration Ratios, A, as a Fraction of Gravitational Acceleration Shown Thus: —— 0.03 ——** **A2 — 1.6**


CANADA
MILES
0.03
0.03
3
2
0.06
0.03
0.01
0
0.01
0.03
0.03
3
2
0.03
0.03
0.06
3
2
0.06
2
2
0.06
3
1

Zone	3	2	1	0
A	0.08 g	0.04 g	0.02 g	0

modified equal area projection

Table **Reference Wind Pressure and Seismic Zoning** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of: 10 yr	25 yr	50 yr	Seismic Zone
British Columbia				
Abbotsford	415	530	620	3
Agassiz	550	715	850	2
Alberni	470	560	630	3
Ashcroft	280	340	385	1
Beatton River	220	265	300	1
Burns Lake	305	355	390	2
Cache Creek	285	340	380	1
Campbell River	455	560	640	3
Carmi	245	315	375	1
Castlegar	225	285	335	0
Chetwynd	320	370	405	1
Chilliwack	475	605	715	2
Cloverdale	455	560	640	3
Comox	445	555	645	3
Courtenay	445	555	645	3
Cranbrook	225	280	325	0
Crescent Valley	225	280	325	0
Crofton	485	565	625	3
Dawson Creek	310	365	400	1
Dog Creek	310	365	400	1
Duncan	485	565	625	3
Elko	270	355	425	0
Fernie	325	410	480	0
Fort Nelson	190	230	265	1
Fort St. John	305	350	385	1
Glacier	240	285	315	1
Golden	270	310	345	0
Grand Forks	265	345	415	1
Greenwood	285	375	445	1
Haney	470	580	675	3
Hope	405	525	625	2
Kamloops	305	360	405	1
Kaslo	225	275	320	0
Kelowna	340	410	470	1
Kimberley	225	280	325	0
Kitimat Plant	380	440	485	3
Kitimat Townsite	380	440	485	3
Langley	455	560	640	3
Lillooet	315	380	430	1
Lytton	310	380	435	1
Mackenzie	245	285	315	1
McBride	275	315	350	1
McLeod Lake	245	285	315	1
Masset	490	565	625	3
Merritt	315	380	430	1
Mission City	465	580	675	3
Montrose	215	285	340	0
Nakusp	235	285	330	0
Nanaimo	470	560	635	3
Nelson	225	280	325	0
New Westminster	440	530	605	3
North Vancouver	440	530	605	3
Ocean Falls	465	535	595	3
100 Mile House	305	355	390	1
Osoyoos	300	405	495	1
Penticton	395	505	590	1
Port Alberni	470	560	630	3

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of: 10 yr	25 yr	50 yr	Seismic Zone
British Columbia (Cont'd)				
Port Hardy	485	565	625	3
Port McNeill	485	565	625	3
Powell River	420	530	620	3
Prince George	210	245	275	1
Prince Rupert	420	485	535	3
Princeton	240	310	365	1
Qualicum Beach	460	560	640	3
Quesnel	250	285	310	1
Revelstoke	240	285	315	2
Richmond	445	535	605	3
Salmon Arm	285	340	380	1
Sandspit	535	615	680	3
Sidney	460	535	595	3
Smithers	315	365	400	2
Smith River	190	245	285	2
Squamish	380	480	560	3
Stewart	325	380	425	3
Taylor	315	365	400	1
Terrace	360	415	460	2
Tofino	540	615	675	3
Trail	170	230	280	0
Ucluelet	540	615	675	3
Vancouver	445	535	605	3
Vernon	315	380	430	1
Victoria	475	560	630	3
Williams Lake	295	340	375	1
Youbou	460	535	595	3
Alberta				
Athabasca	305	360	405	0
Banff	390	440	485	0
Barrhead	315	380	430	0
Beaverlodge	275	320	360	0
Brooks	395	460	515	0
Calgary	395	455	495	0
Campsie	315	380	430	0
Camrose	215	280	335	0
Cardston	740	895	1020	0
Claresholm	655	770	665	0
Cold Lake	310	365	400	0
Coleman	535	665	765	0
Coronation	235	305	365	0
Cowley	730	880	1000	0
Drumheller	315	380	430	0
Edmonton	315	390	450	0
Edson	365	415	460	0
Embarras Portage	305	360	405	0
Fairview	265	310	350	0
Fort McMurray	270	310	350	0
Fort Saskatchewan	315	380	435	0
Fort Vermilion	215	255	285	0
Grande Prairie	370	430	475	0
Habay	200	235	260	0
Hardisty	240	305	365	0
High River	510	585	650	0
Jasper	365	415	455	0

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals)			Seismic Zone
	For Return Periods of:			
	10 yr	25 yr	50 yr	
Alberta (Cont'd)				
Keg River	195	230	260	0
Lac La Biche	310	365	400	0
Lacombe	235	300	350	0
Lethbridge	640	745	825	0
Manning	215	255	290	0
Medicine Hat	395	470	535	0
Peace River	235	285	320	0
Penhold	310	365	405	0
Pincher Creek	705	850	965	0
Ranfurly	235	285	320	0
Red Deer	310	365	405	0
Rocky Mountain House	265	310	350	0
Slave Lake	280	330	370	0
Stettler	240	305	365	0
Stony Plain	320	395	450	0
Suffield	430	510	575	0
Taber	575	670	745	0
Turner Valley	510	590	650	0
Valleyview	355	415	465	0
Vegreville	255	310	355	0
Vermilion	230	275	310	0
Wagner	280	330	370	0
Wainwright	245	310	360	0
Wetaskiwin	240	305	360	0
Whitecourt	320	380	430	0
Wimborne	305	360	405	0
Saskatchewan				
Assiniboia	440	510	570	0
Battrum	490	585	660	0
Biggar	475	580	670	0
Broadview	280	315	340	0
Dafoe	280	330	370	0
Dundurn	395	460	515	0
Estevan	420	495	560	0
Hudson Bay	280	330	370	0
Humboldt	290	350	395	0
Island Falls	450	545	620	0
Kamsack	315	365	400	0
Kindersley	450	560	640	0
Lloydminster	295	360	410	0
Maple Creek	470	560	630	0
Meadow Lake	365	435	495	0
Melfort	260	310	355	0
Melville	320	365	400	0
Moose Jaw	360	415	460	0
Mipawin	270	330	375	0
North Battleford	450	590	705	0
Prince Albert	265	325	380	0
Qu'Appelle	340	385	420	0
Regina	340	385	420	0
Rosetown	470	560	635	0
Saskatoon	360	425	480	0
Scott	435	555	650	0
Strasbourg	335	385	425	0
Swift Current	455	545	620	0

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of: 10 yr	25 yr	50 yr	Seismic Zone
Saskatchewan (Cont'd)				
Uranium City	375	435	490	0
Weyburn	385	440	485	0
Yorkton	315	365	400	0
Manitoba				
Beausejour	305	360	405	0
Boissevain	435	510	570	0
Brandon	375	435	490	0
Churchill	480	570	645	0
Dauphin	315	365	400	0
Flin Flon	415	510	580	0
Gimli	300	360	410	0
Island Lake	365	415	455	0
Lac du Bonnet	280	330	370	0
Lynn Lake	470	560	635	0
Morden	395	460	515	0
Neepawa	335	395	440	0
Pine Falls	285	340	380	0
Portage la Prairie	360	415	465	0
Rivers	360	415	460	0
St. Boniface	350	405	450	0
St. Vital	350	405	450	0
Sandilands	310	365	400	0
Selkirk	325	380	425	0
Split Lake	510	590	650	0
Steinbach	310	365	405	0
Swan River	295	340	380	0
The Pas	350	415	465	0
Thompson	485	565	625	0
Transcona	350	405	450	0
Virden	355	415	465	0
Whiteshell	280	330	370	0
Winnipeg	350	405	450	0
Ontario				
Ailsa Craig	395	480	550	1
Ajax	430	510	575	1
Alexandria	305	360	405	2
Alliston	220	280	330	1
Almonte	295	360	410	2
Ansonville	305	360	405	1
Armstrong	205	240	260	0
Arnprior	275	330	370	2
Atikokan	200	235	260	0
Aurora	305	380	440	1
Bancroft	230	280	325	1
Barrie	210	280	335	1
Barriefield	350	415	465	1
Beaverton	240	305	360	1
Belleville	320	380	430	1
Belmont	350	435	505	1
Bowmanville	460	535	595	1
Bracebridge	190	245	285	1
Bradford	240	305	365	1
Brampton	315	380	435	1
Brantford	310	365	400	1

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
Ontario (Cont'd)				
Brighton	415	485	540	1
Brockville	315	380	430	2
Brooklin	385	460	520	1
Burk's Falls	200	255	295	1
Burlington	360	415	460	2
Caledonia	315	365	400	2
Cambridge	265	310	350	1
Campbellford	290	360	415	1
Camp Borden	215	280	335	1
Cannington	245	310	360	1
Carleton Place	295	360	410	2
Cavan	310	380	435	1
Centralia	375	455	525	1
Chapleau	190	235	270	1
Chatham	320	380	430	1
Chelmsford	285	375	450	1
Chesley	330	410	475	1
Clinton	375	455	525	1
Coboconk	225	280	325	1
Cobourg	465	535	595	1
Cochrane	260	310	350	1
Colborne	440	510	565	1
Collingwood	255	325	385	1
Cornwall	300	360	410	2
Corunna	350	415	465	1
Deep River	200	235	260	2
Deseronto	320	380	430	1
Dorchester Sta	330	410	480	1
Dorion	250	285	310	1
Dresden	320	380	430	1
Dryden	200	235	260	0
Dunbarton	430	510	575	1
Dunnville	335	385	425	2
Durham	310	380	435	1
Dutton	340	410	470	1
Earlton	315	390	450	1
Edison	200	235	260	0
Elmvale	235	305	365	1
Embro	330	410	475	1
Englehart	290	360	415	1
Espanola	280	360	420	1
Exeter	375	455	525	1
Fenelon Falls	250	310	355	1
Fergus	260	310	355	1
Fonthill	335	385	425	2
Forest	390	460	520	1
Fort Erie	365	415	460	2
Fort Frances	200	235	260	0
Gananoque	350	415	465	1
Georgetown	275	330	375	1
Geraldtown	200	235	260	0
Glencoe	310	380	435	1
Goderich	395	480	550	1
Gore Bay	300	350	390	1
Graham	205	240	260	0
Gravenhurst	190	245	285	1
Grimsby	365	415	460	2

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
Ontario (Cont'd)				
Guelph	250	295	325	1
Guthrie	215	280	335	1
Hagersville	335	385	425	1
Haileybury	315	380	435	1
Haliburton	190	235	275	1
Hamilton	365	415	460	2
Hanover	335	410	475	1
Hastings	290	360	415	1
Hawkesbury	310	365	405	2
Hearst	200	245	280	1
Honey Harbour	255	325	385	1
Hornepayne	190	235	270	0
Huntsville	190	240	280	1
Ingersoll	330	410	475	1
Jarvis	330	380	425	1
Jellicoe	200	235	260	0
Kapuskasing	230	275	310	1
Kemptville	295	360	410	2
Kenora	200	235	260	0
Killaloe Sta	235	285	320	1
Kincardine	400	480	545	1
Kingston	350	415	465	1
Kinmount	200	255	295	1
Kirkland Lake	295	360	410	1
Kitchener	275	330	370	1
Lakefield	265	325	380	1
Landsdowne House	240	285	315	0
Leamington	355	415	465	1
Lindsay	265	325	380	1
Lion's Head	330	410	475	1
Listowel	340	410	470	1
London	365	455	535	1
Lucan	395	480	555	1
Maitland	315	380	430	2
Markdale	285	360	415	1
Martin	205	240	260	0
Matheson	300	360	410	1
Mattawa	245	285	315	2
Midland	255	325	385	1
Milton	320	380	430	1
Milverton	310	380	435	1
Minden	190	240	275	1
Mississauga	370	435	495	1
Mitchell	350	435	505	1
Moosonee	195	230	260	1
Morrisburg	300	360	410	2
Mount Forest	290	360	410	1
Muskoka Airport	190	245	285	1
Nakina	200	235	260	0
Napanee	320	380	430	1
Newcastle	460	535	595	1
New Liskeard	315	380	435	1
Newmarket	260	325	385	1
Niagara Falls	330	380	425	2
North Bay	260	300	340	2
Norwood	290	360	415	1
Oakville	375	435	490	1

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of: 10 yr	25 yr	50 yr	Seismic Zone
Ontario (Cont'd)				
Orangeville	250	310	355	1
Orillia	190	250	300	1
Oshawa	430	510	575	1
Ottawa	295	360	410	2
Owen Sound	330	410	475	1
Pagwa River	190	240	275	0
Paris	310	365	405	1
Parkhill	400	480	545	1
Parry Sound	245	325	395	1
Pembroke	215	255	290	2
Penetanguishene	255	325	385	1
Perth	295	360	410	2
Petawawa	195	230	260	2
Peterborough	290	360	415	1
Petrolia	350	415	465	1
Picton	375	435	490	1
Plattsville	295	360	410	1
Point Alexander	200	235	260	2
Porcupine	275	330	375	1
Port Burwell	345	415	470	1
Port Colborne	365	415	455	2
Port Credit	370	435	495	1
Port Dover	360	415	465	1
Port Elgin	395	480	550	1
Port Hope	465	535	595	1
Port Perry	310	380	435	1
Port Stanley	340	410	470	1
Prescott	315	380	430	2
Princeton	300	360	410	1
Raith	205	240	260	0
Red Lake	220	255	285	0
Renfrew	260	310	350	2
Ridgeway	365	415	455	2
Rockland	300	360	410	2
St. Catharines	365	415	460	2
St. Marys	350	435	505	1
St. Thomas	330	410	475	1
Sarnia	350	415	465	1
Sault Ste. Marie	320	365	400	1
Schreiber	250	285	310	1
Seaforth	375	455	525	1
Simcoe	330	380	425	1
Sioux Lookout	205	240	260	0
Smiths Falls	295	360	410	2
Smithville	335	385	425	2
Smooth Rock Falls	235	285	320	1
Southampton	380	460	525	1
South Porcupine	275	330	375	1
South River	230	280	325	1
Stirling	280	345	400	1
Stratford	335	410	475	1
Strathroy	355	435	500	1
Streetsville	350	415	465	1
Sturgeon Falls	255	310	355	1
Sudbury	290	390	465	1
Sundridge	230	280	325	1
Tavistock	340	410	475	1

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 – 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
Ontario (Cont'd)				
Thamesford	330	410	475	1
Thedford	405	485	545	1
Thunder Bay	250	285	310	1
Tillsonburgh	310	380	435	1
Timagami	275	330	375	1
Timmins	255	310	355	1
Toronto	390	460	520	1
Trenton	350	415	465	1
Trout Creek	235	285	320	1
Trout Lake	335	385	425	0
Uxbridge	285	360	415	1
Vanier	295	360	410	2
Vittoria	355	415	465	1
Walkerton	355	435	500	1
Wallaceburg	320	380	430	1
Waterloo	275	330	370	1
Watford	340	410	470	1
Wawa	240	275	305	1
Welland	330	380	425	2
West Lorne	345	415	470	1
Whitby	430	510	575	1
White River	200	235	260	0
Wiarton	330	410	475	1
Windsor	290	350	395	1
Wingham	350	435	505	1
Woodstock	305	380	435	1
Wyoming	350	415	465	1
Quebec				
Acton Vale	235	285	320	2
Alma	235	285	320	2
Amos	240	285	320	1
Ancienne Lorette	385	460	520	3
Arvida	250	310	355	3
Asbestos	260	310	350	2
Aylmer	295	360	410	2
Bagotville	265	325	380	3
Baie Comeau	545	665	760	3
Beaconsfield	315	365	400	2
Beauport	385	460	520	3
Bedford	305	360	405	2
Beloeil	280	330	370	2
Brossard	310	365	400	2
Buckingham	305	360	405	2
Cacouna	410	510	585	3
Campbell's Bay	235	285	320	2
Camp Valcartier	385	460	520	2
Chicoutimi	250	310	355	3
Coaticook	270	330	375	2
Contrecoeur	270	310	345	2
Cowansville	305	360	405	2
Dolbeau	215	255	290	2
Dorval	315	365	400	2
Drummondville	240	285	320	2

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 – 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
Quebec (Cont'd)				
Farnham	310	365	405	2
Fort Chimo	535	640	725	1
Fort Coulonge	240	285	315	2
Gagnon	365	415	455	1
Gaspé	815	950	1060	1
Gatineau	295	360	410	2
Gentilly	230	275	305	2
Gracefield	240	285	315	2
Granby	265	310	350	2
Harrington Harbour	720	905	1060	1
Havre-St.-Pierre	750	895	1015	2
Hemmingford	305	360	405	2
Hull	295	360	410	2
Iberville	310	365	405	2
Inoucdjouac	630	780	905	1
Joliette	250	295	325	2
Jonquière	255	310	355	3
Kenogami	255	310	355	3
Knob Lake	335	385	425	1
Knowlton	280	330	370	2
Kovik Bay	675	815	930	2
Lachine	315	365	400	2
Lachute	310	365	400	2
Lafleche	310	365	400	2
La Malbaie	395	480	555	3
La Salle	315	365	400	2
La Tuque	200	235	260	2
Laval	315	365	400	2
Lennoxville	235	285	320	2
Léry	310	365	400	2
Les Saules	385	460	520	3
Lévis	385	460	520	3
Loretteville	385	460	520	3
Louiseville	215	255	285	2
Magog	260	310	350	2
Malartic	240	285	320	1
Maniwaki	235	275	305	2
Masson	300	360	410	2
Matane	530	660	770	2
Megantic	450	555	645	2
Mont Joli	540	675	785	2
Mont Laurier	240	275	305	2
Montmagny	395	480	555	3
Montréal	315	365	400	2
Montréal Nord	315	365	400	2
Mount Royal	315	365	400	2
Nitchequon	285	330	365	1
Noranda	260	310	350	1
Outremont	315	365	400	2
Percé	825	950	1055	1
Pierrefonds	315	365	400	2
Pincourt	310	365	400	2
Plessisville	260	310	350	2
Pointe-Claire	315	365	400	2
Pointe-Gatineau	295	360	410	2
Port Alfred	250	310	355	3
Port Cartier	670	800	905	3
Poste-de-la-Baleine	635	745	830	0

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of: 10 yr	25 yr	50 yr	Seismic Zone
Quebec (Cont'd)				
Préville	310	365	400	2
Québec	385	460	520	3
Richmond	235	280	320	2
Rimouski	480	580	665	3
Rivière-du-Loup	410	505	585	3
Roberval	215	255	285	2
Rock Island	300	360	410	2
Rosemere	315	365	400	2
Rouyn	260	310	350	1
Ste.-Agathe-des-Monts	270	310	345	2
Ste.-Anne-de-Bellevue	310	365	400	2
St.-Canut	280	330	370	2
St.-Félicien	220	255	285	2
Ste.-Foy	385	460	520	3
St.-Hubert	310	365	400	2
St.-Hubert-de-Temiscouata	410	505	585	3
St.-Hyacinthe	270	310	345	2
St.-Jean	310	365	405	2
St.-Jérôme	285	330	365	2
St.-Jovite	255	295	325	2
St.-Lambert	315	365	400	2
St.-Laurent	315	365	400	2
St.-Nicholas	365	435	495	3
Schefferville	335	380	425	1
Senneterre	240	285	315	1
Sept-Îles	685	815	920	3
Shawinigan	195	230	260	2
Shawville	260	310	350	2
Sherbrooke	205	255	290	2
Sillery	385	460	520	3
Sorel	240	285	315	2
Sutton	305	360	405	2
Tadoussac	400	495	570	3
Témiscaming	240	285	315	2
Thetford Mines	365	435	495	2
Three Rivers	215	255	285	2
Thurso	305	360	405	2
Val-d'Or	240	285	315	1
Valleyfield	310	365	405	2
Varennes	280	330	370	2
Verchères	270	310	345	2
Verdun	315	365	400	2
Victoriaville	260	310	350	2
Ville d'Anjou	315	365	400	2
Ville-Marie	300	360	410	1
Waterloo	260	310	350	2
Westmount	315	365	400	2
Windsor Mills	235	285	320	2
New Brunswick				
Alma	380	480	565	1
Bathurst	335	410	475	2
Campbellton	375	455	525	2

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
New Brunswick (Cont'd)				
Chatham	290	360	415	2
Edmundston	300	380	440	2
Fredericton	295	360	410	2
Gagetown	360	455	535	2
Grand Falls	285	360	415	2
Moncton	460	560	640	1
Oromocto	355	435	500	2
Sackville	410	510	585	1
Saint John	385	460	520	2
St. Stephen	450	535	600	2
Shippegan	515	615	690	1
Woodstock	275	330	370	2
Nova Scotia				
Amherst	410	510	585	1
Antigonish	410	485	545	1
Bridgewater	405	505	585	1
Canso	485	565	625	1
Dartmouth	400	505	590	1
Debert	390	480	555	1
Digby	395	480	550	2
Greenwood	365	455	535	1
Halifax	400	505	590	1
Kentville	355	455	535	1
Liverpool	435	530	610	1
Lockeport	445	535	605	1
Louisburg	515	590	650	2
Lunenburg	425	530	615	1
New Glasgow	400	480	545	1
North Sydney	465	535	595	2
Pictou	400	480	545	1
Port Hawkesbury	590	675	740	1
Springhill	385	480	555	1
Stewiacke	390	480	555	1
Sydney	465	535	595	2
Tatamagouche	395	480	550	1
Truro	370	455	530	1
Wolfville	355	455	535	1
Yarmouth	410	495	560	2
Prince Edward Island				
Charlottetown	460	530	590	1
Souris	415	485	540	1
Summerside	520	615	690	1
Tignish	610	700	775	1
Newfoundland				
Argentia	570	670	750	2
Bonavista	515	615	690	2
Buchans	460	535	595	1
Cape Harrison	460	535	595	1
Cape Race	785	935	1050	2
Corner Brook	575	670	745	1
Gander	460	535	595	2

Table **Reference Wind Pressure and Seismic Zoning (Cont'd)** **A2 — 1.7**

Province and Location	Hourly Mean Wind Pressure, (in pascals) For Return Periods of:			Seismic Zone
	10 yr	25 yr	50 yr	
Newfoundland (Cont'd)				
Goose Bay	285	330	365	1
Grand Bank	590	675	740	2
Grand Falls	460	535	595	1
Labrador City	315	365	400	1
Port aux Basques	545	620	675	2
St. Anthony	570	735	870	2
St. John's	605	710	800	2
Stephenville	620	700	770	1
Twin Falls	315	365	400	2
Wabana	560	670	755	1
Wabush Lake	315	365	400	1
Yukon				
Aishihik	285	340	380	3
Dawson	200	235	260	2
Destruction Bay	295	340	380	3
Snag	200	235	260	3
Teslin	190	245	295	3
Watson Lake	190	230	265	3
Whitehorse	280	330	370	3
Northwest Territories				
Aklavik	370	500	610	3
Alert	535	665	770	0
Arctic Bay	395	480	550	2
Baker Lake	420	485	540	0
Cambridge Bay	295	335	360	2
Chesterfield Inlet	440	510	565	0
Clyde	615	765	890	3
Coppermine	330	400	460	1
Coral Harbour	885	1145	1365	1
Eskimo Point	490	570	640	0
Eureka	475	580	670	1
Fort Good Hope	475	640	780	1
Fort Providence	265	310	350	0
Fort Resolution	290	350	395	0
Fort Simpson	300	360	410	1
Fort Smith	300	360	410	0
Frobisher Bay	565	670	750	2
Hay River	265	310	350	0
Holman Island	635	755	855	2
Inuvik	390	525	640	3
Isachsen	680	800	900	1
Mould Bay	475	580	670	2
Norman Wells	410	550	665	1
Nottingham Island	460	560	640	2
Port Radium	380	460	525	1
Rae	345	415	470	0
Rankin Inlet	460	535	595	0
Resolute	520	610	690	2
Resolution Island	845	1065	1235	2
Tungsten	285	365	445	2
Yellowknife	345	415	470	0

Appendix — Across-Wind Loads on Slender Structural Elements — 2 — 2

General — A2 — 2.1

In accordance with Clause 2 — 4.5.4.4, the time-varying across-wind or lift load on a support structure or slender structural element resulting from vortex shedding is treated as a sinusoidally-varying excitation. Definitions of this excitation for structures with constant cross-section and for tapered structures are provided below.

Vortex Shedding Excitation — A2 — 2.2

The time-varying across-wind or lift excitation resulting from the shedding of vortices shall be determined as follows.

a) *Structures with Constant Diameter or Frontal Width*

The vortex shedding induced load per unit length acting at any location x along the structure shall be taken as:

$$F_s(x,t) = 1/2\rho V^2 C_s D \sin[2\pi n_e t]$$

where: ρ = air mass density, taken as 1.29 kg/m³;
C_s = vortex shedding force coefficient for the section as specified in Table A2 — 3.2;
x = co-ordinate describing length or height along the structure, m;
t = time, s

Table — Vortex Shedding Data — A2 — 2.2

Cross-Section	S	C_s
Circular: subcritical $Re < 3 \times 10^5$	0.18	0.71
supercritical and transcritical $Re > 3 \times 10^5$	0.25	0.28
Square:	0.11	0.85
Multi-sided prismatic shapes, rolled structural shapes	0.15	0.85

The Reynolds Number shall be calculated from

$$Re = \frac{VD}{1.5} \times 10^5$$

For evaluating the amplitude of the resulting forced oscillations, $F_s(x,t)$ shall be taken to act in phase along the entire length of the structure or element under consideration.

b) *Tapered Structures*

The vortex shedding induced load per unit length acting at a particular location x along the structure shall be taken as:

$$F_s(x,t) = 1/2\rho V^2 C_s D(x) \sin[2\pi n_3(x)t]$$

where: $D(x)$ = the diameter for a circular member or the frontal width for a prismatic member or structural shape at location x, m

$$n_e(x) = \frac{SV}{D(x)}$$

For evaluating the amplitude of forced oscillations due to vortex shedding at a frequency of $n_e(x)$, the excitation shall be assumed to act in phase over that portion of the structure or structural element over which the diameter or frontal width does not change by more than ± Ω percent from the value D(x). The quantity Ω shall be taken as 10% unless a smaller value can be justified using an approved procedure. The excitation at this frequency over the remainder of the structure shall be taken as zero. The amplitudes of forced oscillation shall be evaluated for vortex shedding excitation applied at every location x along the structure, in each case taking the excitation to act only over parts of the structure for which D remains with the ± Ω percent of D(x).

In evaluating the response of the structure in a particular mode of vibration, the direction of the vortex shedding induced excitation at any location shall be taken in the direction of the motion of the structure at that location.

Structural Response to Vortex Shedding Excitation A2 — 2.3

Resonance in a particular mode of vibration with a natural frequency n_i can result when $n_e = n_i$. For a structure or element with constant D, this occurs at a critical wind speed given as:

$$V_{cr} = \frac{Dn_i}{S}$$

Resonance for a tapered member or structure in a mode of vibration with frequency n_i can occur over a range of V_{cr} determined by the variation of

$$\frac{D(x)\,n_i}{S} \text{ over its entire length}$$

In accordance with Clause 2 — 4.5.4.4, vortex shedding effects for a particular structure or structural element shall be examined for all modes of vibration for which the critical wind speed,

$$V_{cr} \leq 1.24\sqrt{qC_e}$$

The amplitudes of oscillation of the structure due to the excitation presented in Appendix Clause A2 — 2.2 shall be determined from a steady state forced vibration analysis. Such an analysis can be conveniently carried out by examining the vortex shedding induced response associated with the various normal modes of vibration of the structure. Following this approach, the peak displacement of a structure at location x along its length, oscillating sinusoidally in its i^{th} mode of vibration, can be expressed as

$$y_i(x) = a_i\mu_i(x)$$

where a_i = modal coefficient of magnitude of the oscillatory displacement for the i^{th} mode of vibration, m;
$\mu_i(x)$ = amplitude of the mode shape for the i^{th} mode at location x.

The modal coefficient for a particular mode of vibration due to vortex shedding excitation shall be computed as follows:

a) *Structures with Constant Diameter or Frontal Width*

$$a_i = \frac{\rho C_s D^3 \int_0^H |\mu_i(x)|\,dx}{(4\pi S)^2\,\zeta_i\,GM_i}$$

where: H = the height or length of the structure, m;
ζ_i = the structural damping for the i^{th} mode expressed as a ratio of critical damping;
GM_i = the generalized mass for mode i, taken as:

$$\int_0^H m(x)\,\mu_i^2(x)\,dx$$

m(x) = the mass per unit length of the structure at location x, kg/m

b) *Tapered Structures*

The modal coefficient for mode i due to excitation by vortex shedding from the structure at location $x = x_1$ shall be computed as:

$$a_i(x_1) = \frac{\rho C_s D^2(x_1)\int_{x_1-b}^{x_1+b} D(x) \int |\mu_i(x)|\,dx}{(4\pi S)^2\,\zeta_i GM_i}$$

where: b = the length of an adjacent part of the structure for which D(x) is within Ω percent of $D(x_1)$, with Ω as defined in Appendix Clause A2 — 2.2, m.

This magnitude of $a_i(x_1)$ occurs at wind speed $V = n_i D(x_1)/S$.

The modal coefficient $a_i(x_1)$, defined above, shall be computed for all values of $x = x_1$ along the structure, and the maximum value of $a_i(x_1)$ shall be used for estimating the peak vortex shedding induced response of the tapered structure due to resonance in mode i.

Unless experimentally-determined values are available, the damping ratio for support structures and components in all modes of vibration shall be taken as:

ζ = 0.75% for steel and aluminum, and
ζ = 1.5% for concrete.

Having determined the peak response $y_i(x)$ for the i^{th} mode of vibration of the structure, the maximum stresses obtained during an oscillation shall be computed by applying the peak inertia loads acting statically. The magnitude of the maximum load per unit length at location x along the structure shall be taken as:

$$F_i(x) = (2\pi n_i)^2 a_i \mu_i(x) m(x)$$

Thus, for a free-standing cantilevered structure, the maximum bending moment at location $x = x_1$ shall be taken as:

$$M_i(x_1) = (2\pi n_i)^2 a_i \int_{x_1}^{H} \mu_i(x) m(x) (x - x_1) dx$$

The co-ordinate x in this case is measured from the base of the structure.

The dynamic influence of vortex shedding induced excitation for practical design purposes may be conveniently assessed by representing the actual maximum loads given above, by an equivalent simple loading. The shape and magnitude of this loading are selected to give the same value of maximum bending moment at a critical section as the actual load for a paticular mode of vibration.

In the case of uniform cantilever oscillating in its fundamental mode of vibration, the vortex shedding induced loads can be represented by the following uniformly-distributed load, F_s, acting over its entire length:

$$F_s = \frac{0.30}{\zeta} C_s D V_{cr}^2, \text{ N/m}$$

3 Analysis of Bridge Superstructures

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Scope

3—1

This section covers analysis requirements for bridges, including but not limited to the following types:

A/ Slab
B/ Voided Slab
C/ Slab-on-Girders
D/ Shear - Connected Beams
E/ Truss
F/ Arch
G/ Rigid Frame
H/ Bridges Incorporating Wood Beams
I/ Box Girder — Single Cell
J/ Box Girder — Multi-Cell
K/ Box Girder — Multi-Spine

Definitions

3—2

Coupling Rigidity — the contribution from flexural rigidity to the torsional rigidity of an orthotropic plate, D_1 or D_2.

Deck Slab — a solid or voided concrete slab, carrying and distributing wheel loads to the supporting members.

Distortional Warping Stresses — the longitudinal in-plane stresses developed in the cross-section due to restraint against torsion.

Effective Width — a modification of the actual width of a flange or deck for proportioning of members.

Equivalent Width — a modification of the actual width of the superstructure in order that subsequent analysis may best represent actual behaviour.

External Portion of a Bridge — for a solid slab bridge, the outermost 2 m of the transverse cross-section, on either side of the bridge.

— for a voided slab bridge with rectangular or circular voids, that part of the transverse cross-section which is on the outer side of the vertical plane bisecting the outermost void, on either side of the bridge.

— for a slab-on-girder bridge, that part of the transverse cross-section which is on the outer side of the vertical plane midway between the outermost girder and the girder next to it, on either side of the bridge.

— for a multi-cell box girder bridge, that part of the transverse cross-section which is on the outer side of the vertical plane which bisects the area of the outermost cell, on either side of the bridge.

Flexural Rigidity — the bending stiffness of a beam EI, or

— the bending stiffness per unit width or per unit length of an orthotropic plate, D_x or D_y, respectively.

Floor Beams — transverse structural members spanning between longitudinal girders, trusses or arches.

Floor System — that portion of a bridge superstructure including, where present, the deck, floor beams and stringers, which directly supports traffic and transmits loads to longitudinal members.

Grid — a plane assembly of intersecting beams subject to loading perpendicular to the plane formed by the assembly and characterized by the fact that interaction between beams takes place only at their intersections.

Idealization — the representation of a structure or load for purposes of mathematical analysis or model testing.

In-Plane Forces — forces acting along the neutral surfaces of planar elements.

Interface Shear — the shear between the deck and a supporting beam.

Internal Portion of a Bridge — that part of the transverse cross-section contained between the two external portions.

Longitudinal Direction — the direction of traffic flow.

Longitudinal Moment — moment in a longitudinal vertical plane, that is, moment about a transverse axis.

Longitudinal Shear — vertical shear in the longitudinal vertical plane which, at the supports of single span bridges, becomes the support reaction.

Longitudinal Torsion — torsion about a longitudinal axis.

Mathematical Model — a conceptual approximation to a structure for purposes of analysis.

Modification Factor — a factor applicable to highway live loads, as defined in Clause 2—4.3.1.9.

Moment of Inertia — the second moment of area of the cross-section of a component, with respect to a centroidal axis unless specified otherwise.

Orthotropic Plate — a plate having different flexural and torsional rigidities in mutually perpendicular directions.

Radius of Curvature — the plan form radius of a longitudinal line joining the centroids of transverse cross-sections.

Shear-Connected Beams — longitudinal beams placed side by side with the connections between adjacent ones including shear keys or similar devices.

Shallow Superstructure — slab, slab-on-girders, voided slab, and shear-connected beam bridge types, provided that in the case of shear-connected beams, the interconnection of adjacent beams is such as to provide continuity of transverse flexural rigidity across the cross-section.

Skew Angle — for bridges in which the plan form is a parallelogram, the angle obtained by subtracting the acute angle of the parallelogram from 90°.

Skew Span — for bridges in which the plan form is a parallelogram, the distance between the support lines measured along the unsupported edge.

Skew Width — for bridges in which the plan form is a parallelogram, the length of the supported edge.

Span — for bridges with parallel lines of supports, the perpendicular distance between the lines of supports; or

— for curved-in-plan bridges, the length between the supports taken along the line of centroids of cross-sections.

Stringers — longitudinal structural members spanning between floor beams.

Torsional Warping Stresses — the longitudinal in-plane stresses developed in the cross-section of a component due to torsional loading resulting from the partial or full restraint of the associated in-plane movements.

Transverse Direction — the direction perpendicular to the longitudinal direction at any point on the bridge.

Transverse Moment — moment in the transverse vertical plane, that is, moment about a longitudinal axis.

Transverse Shear — vertical shear in the transverse vertical plane.

Transverse Torsion — torsion about a transverse axis.

Width — the perpendicular distance between the unsupported edges of a bridge.

Notation 3—3

Note concerning units: In the formulae used in Section 3, the units in which the various quantities are expressed shall be mutually consistent. If a formula requires that a given quantity or quantities be expressed in particular units, those units are given at the time that the formula is presented.

A_x	the equivalent area per unit width of the transverse section in a voided slab.
A_y	the equivalent area per unit length of the longitudinal section in a voided slab.
A_1	an area as defined in Appendix Table A3 — 2(b)
A_2	an area as defined in Appendix Table A3 — 2(b)
a	the length of the longer side of a rectangular section; or the distance between adjacent transverse bracing members.
B	the width of a bridge; or one-half of the transverse span of a deck slab as defined in Figure 3 — 10.2(a)
B_e	a reduced value of B.
b	the half-width of a bridge; or the length of the shorter side of a rectangular section; or the clear distance between adjacent flanges; or the width of a timber beam as shown in Table A3 — 2(d).
C	the width of deck slab used to represent an edge stiffening beam.
C_f	a correction factor used to adjust the D value for longitudinal moment and longitudinal shear.
D	the load distribution factor.
D_d	the load distribution factor modified for design.
D_x	the longitudinal flexural rigidity per unit width.
D_{xy}	the longitudinal torsional rigidity per unit width.
D_y	the transverse flexural rigidity per unit length.
D_{yx}	the transverse torsional rigidity per unit length.
D_1	the coupling rigidity per unit width.
D_2	the coupling rigidity per unit length.
DLA	dynamic load allowance as defined in Section 2.

E	the modulus of elasticity.
E_c	the modulus of elasticity of concrete.
E_L	the modulus of elasticity of wood in the direction defined in Appendix Figure A3 — 2(b).
E_s	the modulus of elasticity of steel.
E_T	the modulus of elasticity of wood in the direction defined in Appendix Figure A3 — 2(b).
F	a distribution factor for transverse moment, defined in Clause 3 — 7.1.3.1.
F_c	a correction factor defined in Clause 3 — 7.1.2.2.
F_d	a distribution factor for transverse moment, modified for design, defined in Clause 3 — 7.1.3.1.
F_1	a modification factor relating the transverse moment of inertia of a voided slab to the corresponding slab without voids.
F_2	a modification factor relating the torsional moment of inertia of a voided slab to the corresponding slab without voids.
G	the shear modulus.
G_c	the shear modulus of concrete.
G_{LT}	the shear modulus of wood with respect to axes defined in Appendix Figure A3 — 2(b).
G_s	the shear modulus of steel.
H	a ratio as defined in Clause 3 — 7.1.5.2(b).
I	the moment of inertia of the cross-section of a beam.
i	the longitudinal moment of inertia per unit width.
i_o	the longitudinal torsional inertia per unit width.
J	the transverse torsional inertia of a beam.
j	the transverse moment of inertia per unit length.
j_o	the transverse torsional inertia per unit length.
K	a torsional constant for a rectangular section.
L	the span of a bridge, or the span of the curved portion of a bridge.
L_L	a load fraction as defined in Clause 3 — 7.1.5.2(b).
M_x	the longitudinal bending moment per unit width.
M_{xave}	the longitudinal bending moment per unit width, as defined in Clause 3 — 7.1.3.1(a).
M_y	the transverse bending moment per unit length.
N_w	the number of design lanes, defined in Clause 2 — 4.3.1.4.
n	the modular ratio E_s/E_c for steel and concrete; or the modular ratio of girder or beam material to slab material.
P_x	the spacing of longitudinal beams or webs; or the spacing, centre-to-centre, of circular voids in a voided slab.
R	the mean radius of curvature of the curved portion of a bridge which is curved in plan.
S	a spacing dimension as defined in Clause 3 — 7.1.2.1.
SLS I	the serviceability limit state, Type I.
SLS II	the serviceability limit state, Type II.
S_y	the transverse shear rigidity per unit length.
s	the shear area per unit length.
t	the overall thickness of a slab.
t_v	the diameter of a circular void or the height of a rectangular void.
t_w	the minimum thickness of the web section between circular voids; or the thickness of the internal web between rectangular voids.
t_1	the thickness of the top flange of a rectangular voided slab.
t_2	the thickness of the bottom flange of a rectangular voided slab.
ULS	the ultimate limit state.
W_c	the bridge deck width, m.
W_e	the width of a design lane, m.
W_L	a fraction of a wheel load as defined in Clause 3 — 7.1.5.2(b).
α	torsional parameter $= (D_{xy} + D_{yx} + D_1 + D_2)/2\,[D_xD_y]^{0.5}$
γ	$(w_c - 7.5)/9.0$, $\nless 0.0$ and $\ngtr 1.0$.
δ	a parameter defined in Clause 3 — 7.1.5.1.
θ	a flexural parameter $= (b/L)\,[D_x/D_y]^{0.25}$
λ	a parameter defined in Clause 3 — 7.1.2.2.

λ_m	a parameter defined in Clause 3 — 7.1.5.1
λ_s	a parameter defined in Clause 3 — 7.1.6.1
μ	$(W_e - 3.3)/0.6, \ngtr 1.0$
ν	Poisson's ratio

General Requirements 3 — 4

The requirements of Clause 3 — 4 apply to bridges of all types, subject to additions or deletions as given in Clause 3 — 5 for certain types of bridges.

Analysis for Limit States 3 — 4.1

Analysis for all limit states shall be linear elastic. The use of non-linear methods of analysis, unless specifically permitted in other sections of the Code, requires approval.

Analysis for Dynamic Loads 3 — 4.2

The methods of analysis specified in this section relate to static loads. Whenever such methods are to be used for the analysis of a bridge under dynamic loads, these must be converted to equivalent static loads under the dynamic load allowance provisions of Section 2.

Structural Responses 3 — 4.3

With the exceptions and additions given elsewhere in this section for particular types, bridges of all types shall be analysed for the following responses:

a) longitudinal moment
b) transverse moment
c) longitudinal torsion
d) longitudinal shear and support reaction
e) transverse shear
f) deformation

Deformations shall be calculated in accordance with Clause 3 — 4.5.

Factors Affecting Structural Responses 3 — 4.4

In deriving the appropriate structural responses, all relevant factors present in the design, both geometric and non-geometric, shall be taken into account, including, but not limited to, the following:

a) continuity of spans
b) plan geometry, including skewness and curvature
c) edge stiffening
d) longitudinal variation of transverse section
e) transverse variation of longitudinal section
f) diaphragms and cross frames
g) wind bracing
h) interaction of floor system and its support system
i) barrier and parapet walls
j) support conditions other than line support
k) movement of supports, also supports for continuous and skew spans
l) temperature
m) creep and shrinkage
n) secondary force effects
o) elastic shortening
p) construction sequence

The above factors may be taken account of in accordance with Appendix A3 — 1.

Deformations 3 — 4.5

3 — 4.5.1 General

Deformations shall be calculated using the static values of the relevant material properties. Deformations due to prestress shall be taken into account in accordance with the requirements of Clause 3 — 4.10.

3 — 4.5.2 Vertical Deflections Due To Dead Load

Dead load deflections may be calculated according to the beam analogy method specified in Clause 3 — 6.1.2 provided that the requirements of that clause are met. Construction sequence shall be taken into account in accordance with the requirements of Clause 3 — 4.9.

3 — 4.5.3 Vertical Deflections Due To Live Load

In the absence of a more rigorous analysis, live load deflections of a bridge which satisfies the requirements of Clause 3 — 7.1.1 may be determined as follows:

a) Regardless of the transverse position of the live load, the average deflection of the transverse cross-section of the bridge may be determined by treating the bridge as a beam and calculating the deflection of the beam due to flexure.
b) The maximum deflection of any point on the transverse cross-section of a bridge of shallow superstructure may be estimated by using the method of Clause 3 — 7.1.2.2 to determine the load fraction S/D_d and thereafter applying this fraction of the live loads specified in Clause 3 — 7.1.2.2(f) to a longitudinal strip of width S. The maximum deflection at the transverse cross-section of the bridge

may be taken as the deflection of the corresponding point on the longitudinal strip, the strip being treated for this purpose as a simple beam in flexure.

The maximum deflection of any point on the transverse cross-section of a multi-cell box girder bridge may be estimated by the same method, in accordance with the additional requirements of Clause 3 — 7.1.5.1, and of a multi-spine box girder bridge in accordance with the additional requirements of Clause 3 — 7.1.5.2(a).

For multi-spine bridges of steel-concrete composite construction, the maximum deflection may be estimated by applying the fraction of loads given by Clause 3 — 7.1.5.2(b) to one spine, the spine being treated for this purpose as a simple beam.

Diaphragms and Bracing Systems **3 — 4.6**

Diaphragms and bracing systems shall conform to the relevant requirements of Sections 8, 9, 10, and 13, and shall be analysed in accordance with the relevant requirements of Appendix A3 — 1.

Analysis of Deck Slabs **3 — 4.7**

In cases where the empirical and yield line methods of Section 7, in relation to deck slabs, are not applicable, the analysis of deck slabs shall be based upon elastic methods, both in relation to overall behaviour and to local behaviour.

Analysis for Redistribution of Force Effects **3 — 4.8**

The effect of creep and shrinkage on redistribution of force effects shall be considered, and elastic methods of analysis shall be used for this purpose.

Analysis for Accumulation of Force Effects due to Construction Sequence **3 — 4.9**

Accumulation of force effects, due to construction sequence, shall be considered. For calculation of force effects at a particular stage in the construction sequence, elastic methods shall be used and the material properties shall be those appropriate to that stage of construction.

Analysis for Effects of Prestress **3 — 4.10**

The out-of-plane and in-plane force effects arising from prestress shall be taken into account. In the absence of a more rigorous analysis the prestressing force shall be resolved into out-of-plane and in-plane force components, and the out-of-plane component shall be replaced by a statically equivalent out-of-plane loading, which shall be considered along with other relevant loads.

Requirements For Specific Bridge Types **3 — 5**

The general requirements of Clause 3 — 4 are subject to additions or deletions as given in Clause 3 — 5 for the bridge types concerned. In the case of bridges of types other than types A to K of Clause 3 — 1, engineering judgement shall be exercised as to what additions or deletions are applicable.

Slab and Voided Slab **3 — 5.1**

3 — 5.1.1

Structural Responses

a) In the case of dead load, provided that the conditions of Clause 3 — 6.1.1 are met, the bridge need not be analysed for
    i) longitudinal torsion, or
    ii) transverse shear.
b) In the case of live load, provided that the conditions of Clause 3 — 7.1.1 are met, the bridge need not be analysed for
    i) longitudinal torsion, or
    ii) transverse shear.

3 — 5.1.2

Limitation on Size of Voids

If a bridge is to be treated as a voided slab type, the limiting size of the voids shall be as follows:

a) For circular voids, the diameter of the void shall not exceed 80% of the total depth of the slab, and the spacing of the voids, centre to centre, shall not be less than the total depth of the slab.
b) For rectangular voids, the thickness of the web defined by adjacent voids shall be not less than 20% of the total depth of the section, the depth of the void shall not exceed 80% of the total depth of the section, and the transverse width of the void shall not exceed 1½ times its depth.

In cases where the voids do not conform to the requirements of (a) or (b) above, as appropriate, the bridge shall be treated as being of the multi-cell type.

Slab-On-Girders **3 — 5.2**

For the structural responses of slab-on-girder bridges, the following shall apply.

a) The longitudinal moment shall be assumed to be resisted only by the girders, if the structural action is non-composite, and by the girders plus an appropriate portion of the slab if the structural action is composite.

b) The transverse moment shall be assumed to be taken by the slab; the action of cross-bracing or diaphragms, if present, may be taken into account. If the slab is designed in accordance with the provisions of Section 7, the requirements of that section in relation to transverse moment shall be met, and are sufficient.

c) In the case of dead load, provided that the conditions of Clause 3 — 6.1.1 are met, the bridge need not be analysed for:
   i) longitudinal torsion, or
   ii) transverse shear.

d) In the case of live load, provided that the conditions of Clause 3 — 7.1.1 are met, the bridge need not be analysed for:
   i) longitudinal torsion, or
   ii) transverse shear.

Shear-Connected Beams **3 — 5.3**

For the structural responses of shear-connected beam bridges, the following shall apply.

a) If the beams are connected in such a manner as to provide continuity of transverse flexural rigidity across a transverse section, the bridge shall be analysed as slab, voided slab, or slab-on-girders, as appropriate, except that an analysis for transverse shear shall always be performed for live loads.
   If the beams are connected in such a manner as not to provide continuity of transverse flexural rigidity across a transverse cross-section, then (b) and (c) below shall apply.

b) In the case of dead load, provided that the conditions of Clause 3 — 6.1.1 are met, the bridge need not be analysed for:
   i) transverse moment,
   ii) longitudinal torsion, or
   iii) transverse shear.

c) In the case of live load, provided that the conditions of Clause 3 — 7.1.1 are met, the bridge need not be analysed for:
   i) transverse moment,
   ii) longitudinal torsion.

Truss and Arch **3 — 5.4**

For the structural responses of truss and arch bridges the following shall apply.

a) The floor system shall be analysed for the structural responses listed in Clause 3 — 4.3 with the exception of the following, analysis for which shall be optional:
   i) longitudinal moment,
   ii) longitudinal torsion, and
   iii) longitudinal shear.

b) Axial forces for each member of the truss or arch shall be considered.

c) Strength of connections, overall stability, and stability of the compression chord of a pony truss shall be considered.

Rigid Frame **3 — 5.5**

In rigid frame bridges, the analysis for structural responses required by Clause 3 — 4.3 shall include in-plane forces induced by frame action.

Bridges Incorporating Longitudinal Wood Beams **3 — 5.6**

For structural responses of bridges incorporating longitudinal wood beams, the following shall apply.

a) In the case of dead load, provided that the conditions of Clause 3 — 6.1.1 are met, the bridge need not be analysed for:
   i) transverse moment,
   ii) longitudinal torsion, or
   iii) tranverse shear.

b) In the case of live load, provided that the conditions of Clause 3 — 7.1.1 are met, the bridge need not be analysed for:
   i) transverse moment, except for deck-on-stringer types,
   ii) longitudinal torsion, or
   iii) transverse shear, except for prestressed wood decks.

Box Girder **3 — 5.7**

For the structural responses of all box girder bridges the following shall apply.

a) For dead load, all of the structural responses listed in Clause 3 — 4.3 shall be considered. If the transverse cross-section of the bridge is unsymmetrical, engineering judgement shall be exercised as to whether to include torsional warping and distortional warping effects in the analysis.

b) For live load, all of the structural responses listed in Clause 3 — 4.3 shall be considered; torsional warping and distortional warping effects shall be included in the analysis.

Dead Load 3 — 6

Simplified Methods of Analysis 3 — 6.1

3 — 6.1.1 General

For dead load analysis, the beam analogy method given in Clause 3 — 6.1.2 may be used for bridges satisfying the following conditions:

a) the width is constant;

b) the support conditions are closely equivalent to line support, both at the ends of the bridge and, in the case of multi-span bridges, at intermediate supports;

c) for slab bridges and slab-on-girder bridges with shored construction, the skew angle does not exceed 20°;

d) the radius of curvature, span and width satisfy the requirements of Appendix A3 — 1(b)(ii);

e) for a bridge having longitudinal girders and an overhanging deck slab, the overhang does not exceed 60% of the mean spacing between the longitudinal girders, and, also, is not more than 1.8 m.

In a case where these conditions are not fully met, engineering judgement shall be exercised as to whether the bridge being designed or evaluated meets them sufficiently closely for the simplified method to be applicable.

3 — 6.1.2 The Beam Analogy Method

For a bridge satisfying the requirements of Clause 3 — 6.1.1, it is permitted to treat the whole of the bridge superstructure, or of part of the bridge superstructure contained between two parallel vertical planes running in the longitudinal direction, as a beam, for the purpose of evaluating longitudinal moments and longitudinal shears due to dead load. The dead load of the cast deck and superimposed dead load shall be distributed in such a manner as to satisfy overall equilibrium.

Any element of superstructure in which the load is carried preponderantly by flexure in one direction may be analysed as a beam, including but not limited to the following:

a) a slab supported along two opposite edges only;

b) a slab supported along three or four edges, having a width more than 2.0 times the span;

c) floor beams and associated portions of a deck;

d) stringers and associated portions of a deck.

Refined Methods of Analysis 3 — 6.2

In cases where the requirements of Clause 3 — 6.1 are not met, a refined method in accordance with Clause 3 — 8 shall be used. In cases where the requirements of Clause 3 — 6.1 are met, a refined method in accordance with Clause 3 — 8 may be used.

Live Load 3 — 7

Simplified Methods of Analysis 3 — 7.1

3 — 7.1.1 General

For live load analysis, the simplified methods of Clauses 3 — 7.1.2 to 3 — 7.1.7 may be used for the bridge types identified, provided that the bridges concerned satisfy the following conditions:

a) the width is constant;

b) the support conditions are closely equivalent to line support, both at the ends of the bridge and, in the case of multi-span bridges, at intermediate supports;

c) the skew angle does not exceed 20°;

d) the radius of curvature, span and width satisfy the requirements of Appendix A3 — 1(b) (ii);

e) a solid or voided slab is of substantially uniform depth across a transverse section, or tapered in the vicinity of a free edge provided that the length of the taper in the transverse direction does not exceed 2.5 m;

f) the total longitudinal flexural rigidity of a transverse cross-section of the bridge remains the same over at least the central 50% of each span;

g) for slab-on-girder bridges, longitudinal girders are of equal flexural rigidity and equally spaced, or with variations from the mean of not more than 10% in each case;

h) for slab-on-girder bridges having an overhanging deck slab, the overhang does not exceed 60% of the mean spacing between girders and, also, is not more than 1.8 m; further, the outermost extremity of a design lane shall not be more than 1.0 m outside of the centreline of the outermost girder.

For a continuous span bridge, the provisions of Appendix A3—1(a) shall apply.

In a case where the above conditions (a) to (h) are not fully met, engineering judgement shall be exercised as to whether the bridge being designed or evaluated meets them sufficiently closely for the simplified method to be applicable.

For shear-connected beam bridges in which the interconnection of adjacent beams is such as to provide continuity of transverse flexural rigidity across the cross-section, the bridge shall be treated as a shallow superstructure type and Clauses 3—7.1.2, 3—7.1.3 and 3—7.1.4 shall apply.

For shear-connected beam bridges in which the interconnection of adjacent beams is such as not to provide continuity of transverse flexural rigidity across the cross-section, analysis for longitudinal bending moments and longitudinal shears shall be by the same methods as are made available for multi-spine box girder bridges in Clauses 3—7.1.5.2(a) and 3—7.1.6.2(a), respectively.

3—7.1.2 Longitudinal Bending Moments in Shallow Superstructures

3—7.1.2.1 Longitudinal Bending Moments for Ultimate Limit States and Serviceability Limit States Type II

For design or evaluation of a bridge satisfying the requirements of Clause 3—7.1.1, the following method may be used for obtaining governing live load moments in the internal and external portions of the bridge.

a) An inital value of D_d shall be selected from Table 3—7.1.2.1 for the type of bridge and number of design lanes concerned. This value shall be used for both internal and external portions.

Table **Initial Values of D_d in Metres for Longitudinal Moment for ULS and SLS II** **3—7.1.2.1**

Bridge Type	Number of Design Lanes 1	2	3	4 or more
A. Slab	1.75	2.00	2.25	2.50
B. Voided Slab	1.75	2.00	2.25	2.50
C. Slab-on-girders, including steel girders with wood decking	1.90	1.90	2.05	2.15
H. Bridges incorporating longitudinal wood beams, except for nail-laminated decks and wood-concrete	1.80	1.80	1.85	1.90
composites	1.90	1.90	2.05	2.15
J. Box Girder, Multi-Cell		1.90	2.15	2.50
K. Box Girder, Multi-Spine		2.00	2.10	2.30

Notes:

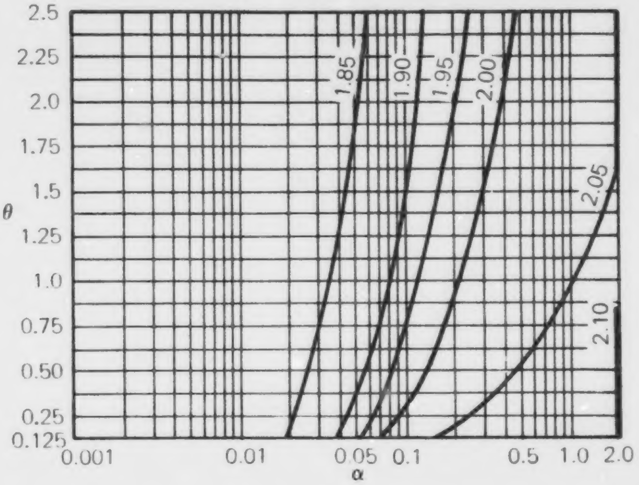
i) Modification factors given in Table 2—4.3.1.9 are implicit in the values for D_d given in the above table.

ii) The values of D_d relate to ultimate limit state and serviceability limit states Type II.

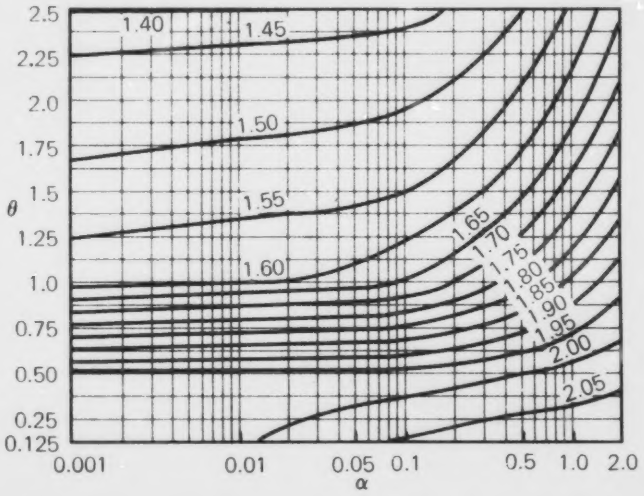
iii) The values of D_d for bridges of types J and K are for use in conjunction with Clauses 3—7.1.5.1 and 3—7.1.5.2, respectively.

Figure **Charts for ULS and SLS II Cases in Two-Lane Bridges** **3 – 7.1.2.1(a)**

D. for external portions, metres



D. for internal portions, metres



C_f, percent

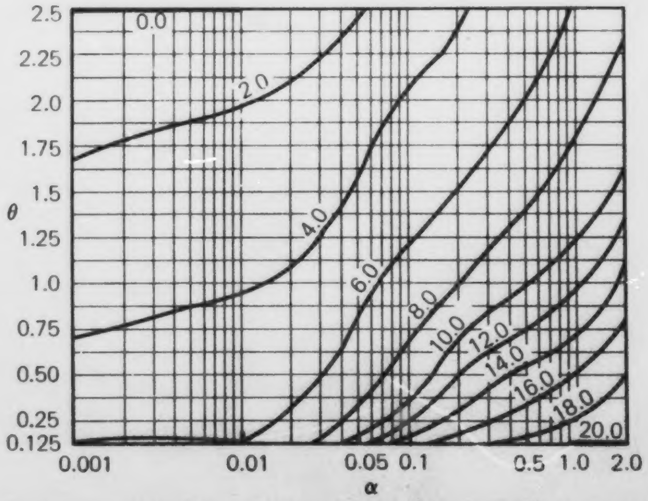
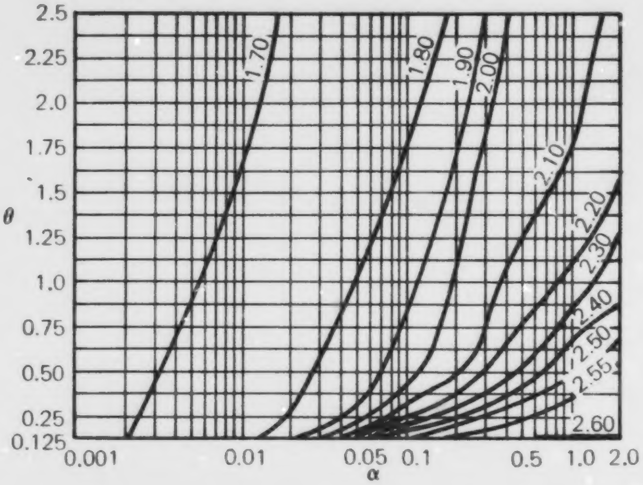


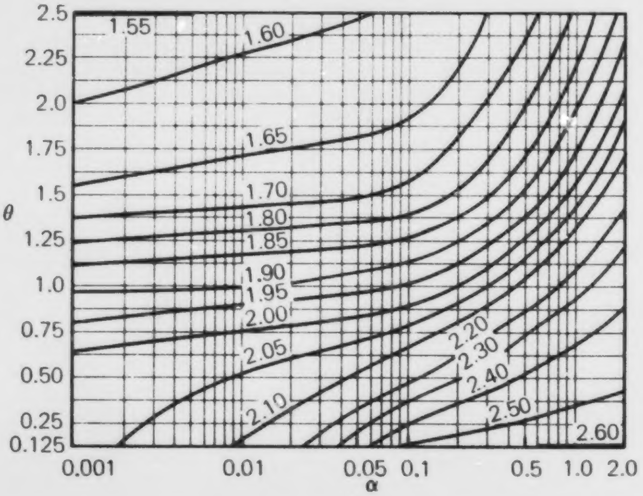
Figure **Charts for ULS and SLS II Cases in Three-Lane Bridges** **3 — 7.1.2.1(b)**

D. for external portions, metres



Ta

D. for internal portions, metres



C_f, percent

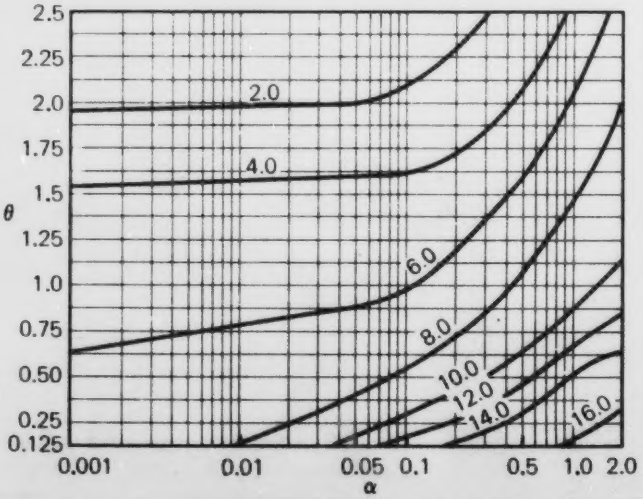
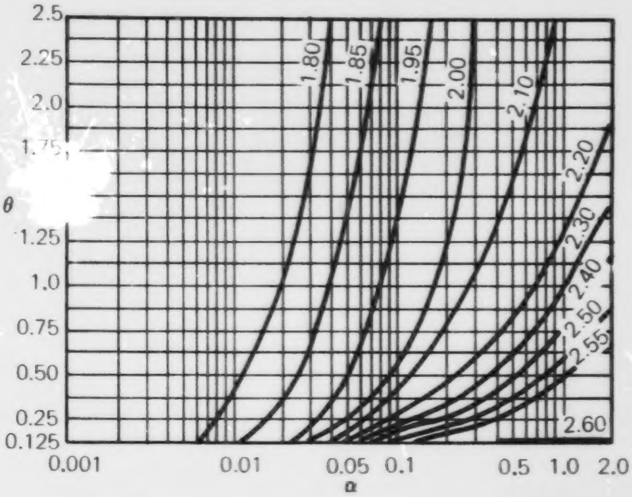
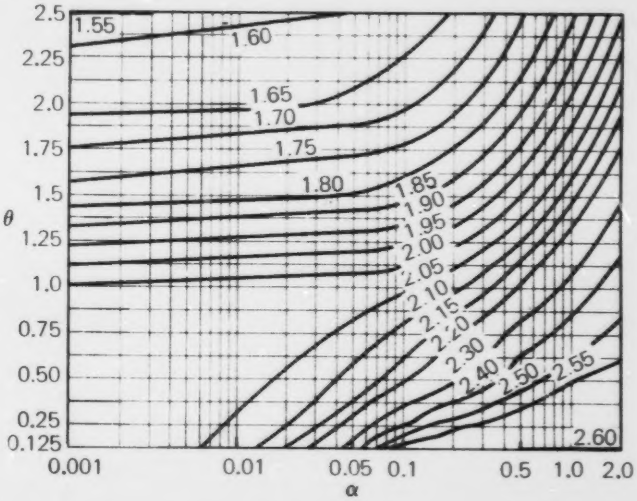


Figure **Charts for ULS and SLS II Cases in Four-Lane Bridges** **3 — 7.1.2.1(c)**

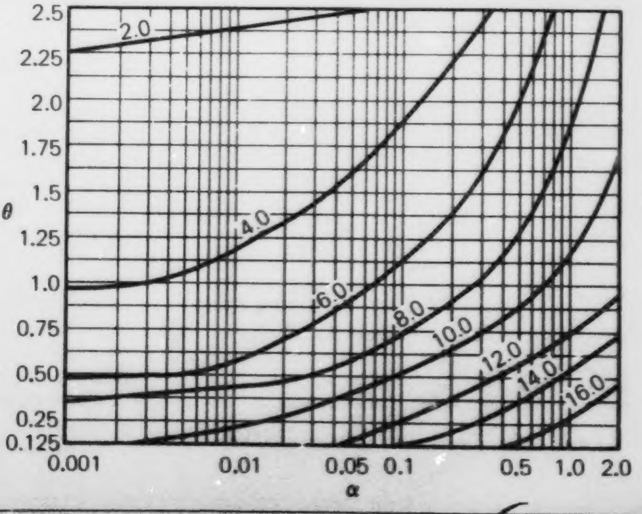
D, for external portions, metres



D, for internal portions, metres



C_f, percent



b) The initial load fraction S/D_d shall be calculated, where:
   S = the actual spacing of longitudinal girders, in metres; or
   = the spacing of webs, in metres, in the case of voided slabs or cellular structures; or
   = 1 m in the case of solid slabs and transversely prestressed laminated wood bridges.

c) Longitudinal bending moment diagrams shall be obtained, treating the bridge as a beam, for two load cases: the first load case comprises one line of wheel loads specified in Section 2, Figure 2—4.3.1.2(a), and the second load case comprises one-half of the lane load specified in Section 2, Figure 2—4.3.1.2(b). The governing bending moments from these two diagrams shall be multiplied by the appropriate load fraction S/D_d and also by the factor (1 + DLA), to give the design live load longitudinal bending moment for a width S of the cross-section, for both internal and external portions of the bridge, respectively.

d) The design live load longitudinal bending moments, as obtained in (c) above, shall be used, without modification factors, for the initial proportioning of both internal and external portions of the superstructure.

e) Following the initial proportioning of the structure, the final values of D_d shall be calculated, for the internal and external portions of the bridge, as follows:
   i) Values of D_x, D_y, D_{xy}, D_{yx}, D_1 and D_2 shall be calculated using the formulae given in Appendix A3—2.
   ii) Values of α, θ and μ shall be calculated from:

$$\alpha = \frac{D_{xy} + D_{yx} + D_1 + D_2}{2(D_x D_y)^{0.5}}$$

$$\theta = \frac{b}{L}\left[\frac{D_x}{D_y}\right]^{0.25}$$

$$\mu = \frac{W_e - 3.3}{0.6}, \text{ but not greater than } 1.0$$

where: b = half-width of the bridge,
L = span of the bridge, and
W_e = width of a design lane in metres, calculated in accordance with Clause 2—4.3.1.4.

   iii) Corresponding to the values of α and θ, values of D for internal and external portions, and C_f shall be obtained from the relevant graphs of Figures 3—7.1.2.1.(a) to (c), for bridges of two, three, or four or more design lanes, respectively. For one-lane bridges the values of D and C_f shall be obtained from Figure 3—7.1.2.2 (a), without making any distinction between external and internal portions.
   iv) The final values of D_d for internal and external portions shall be obtained by using the appropriate values of D and C_f, from the following equation:

$$D_d = D\left[1 + \frac{\mu C_f}{100}\right]$$

f) The governing live load bending moments for internal and external portions of the bridge shall be calculated using the final values of D_d and the procedure given in (c), without using modification factors.

For evaluation of existing bridges satisfying the requirements of Clause 3—7.1.1, the live load longitudinal bending moment shall be calculated using steps (e) and (f) above, for a loading comprising one-half of the relevant loads shown in Section 2, Figures 2—4.3.1.2 and 2—4.3.1.3.

Notwithstanding the above, for existing longitudinally-laminated wood bridges in which the laminates are connected only by means of nails, the value of D_d shall be taken as 0.85 m for both internal and external portions.

3—7.1.2.2 Longitudinal Bending Moments for Serviceability Limit States Type I

For a bridge satisfying the requirements of Clause 3—7.1.1, the following methods may be used for obtaining governing longitudinal moments.

a) Using the properties of the structure, values of α, θ and μ shall be calculated according to the method given in Clause 3—7.1.2.1(e) (ii).

b) Corresponding to the values of α and θ, values of D and C_f shall be obtained from the relevant graphs of Figure 3—7.1.2.2(a) to (d).

c) A value of D_d shall be obtained by using the appropriate values of D and C_f from the following equation:

$$D_d = D\left[1 + \frac{\mu C_f}{100}\right]$$

d) For bridges having two or more design lanes, when the distance of the centreline of the outside line of wheels from the nearer longitudinal free edge of the bridge is more than 1 m, the D_d value may be modified as follows.

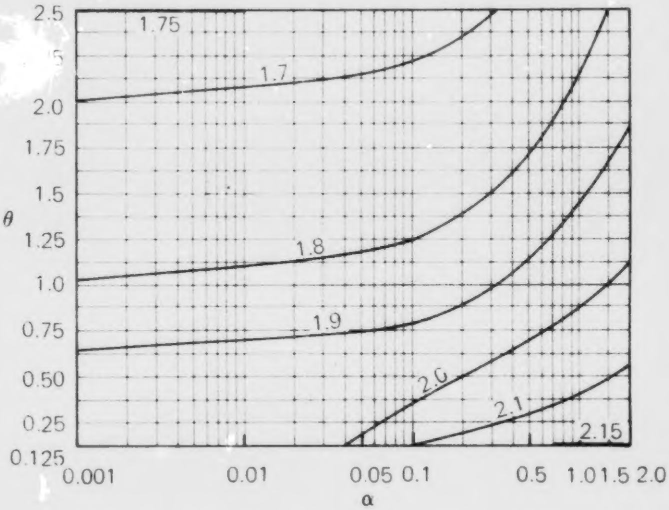
i) The value of λ shall be calculated from:

$$\lambda = \frac{EI}{LD_x}\left[\frac{D_x}{D_y}\right]^{0.25}$$

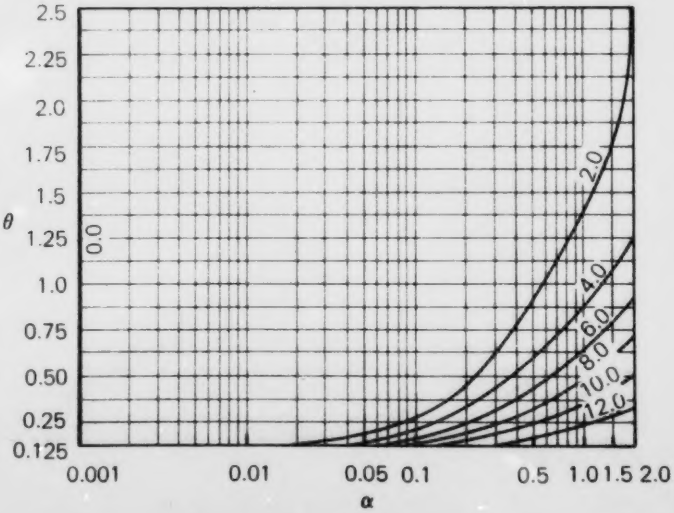
where EI is the flexural rigidity of that part of the cross-section, on one side,

Figure **Charts for ULS, SLS II and SLS I Cases in One-Lane Bridges** **3 – 7.1.2.2(a)**

D in metres



C_f, percent



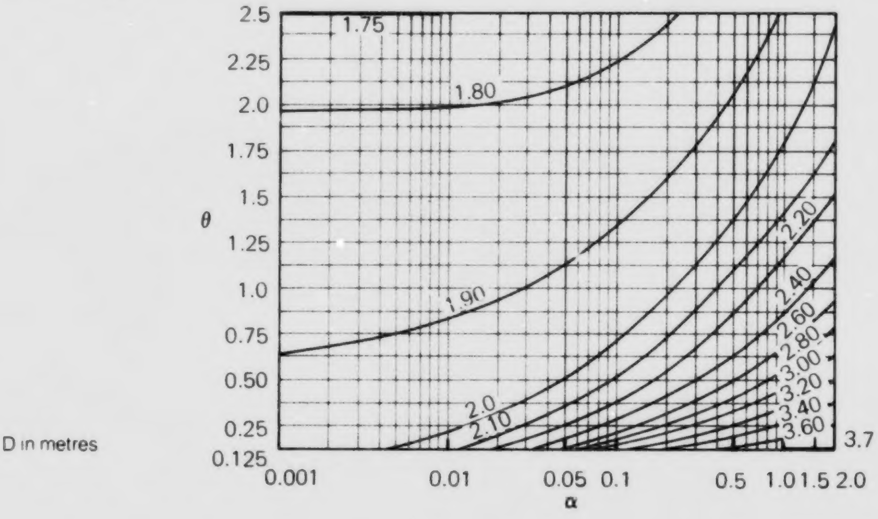
which lies outside of a point 1.0 m outer of the centreline of the outside line of wheels, as shown in Figure A3—1(c).

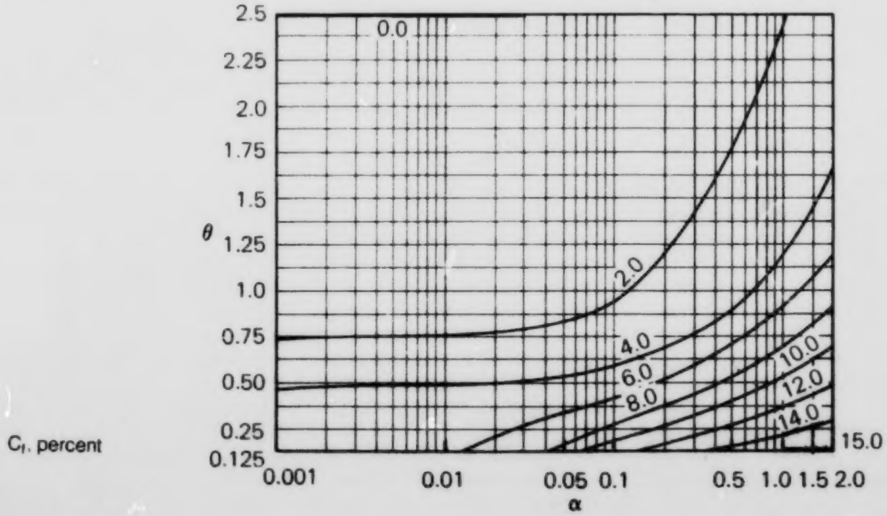
- ii) Corresponding to the values α, θ and λ, the value of a correction factor F_c shall be obtained from the relevant charts of Figure 3—7.1.2.2(e).
- iii) The modified value of D_d shall be obtained by dividing the value D_d in (c) above, by the value of the correction factor F_c.

e) The value of the load fraction S/D_d shall be calculated, where S is as defined in Clause 3—7.1.2.1(b).

f) The longitudinal bending moment shall be calculated treating the bridge as a beam for one line of wheel loads, specified in Section 2, Figure 2—4.3.1.2(a) and in accordance with Clause 2—4.3.1.7. The bending moments, thus obtained, shall be multiplied by the load fraction S/D_d, as obtained in (e) above, and by the factor (1+DLA), to give the live load longitudinal bending moments for a width S of

Figure **Charts for SLS I Cases in Two-Lane Bridges** **3 — 7.1.2.2(b)**





the cross-section, where DLA is the appropriate dynamic load allowance for a single vehicle.

Notwithstanding the above, for existing longitudinally-laminated wood bridges in which the laminates are connected only by means of nails, the value of D_d shall be taken as 0.85 m for both internal and external portions.

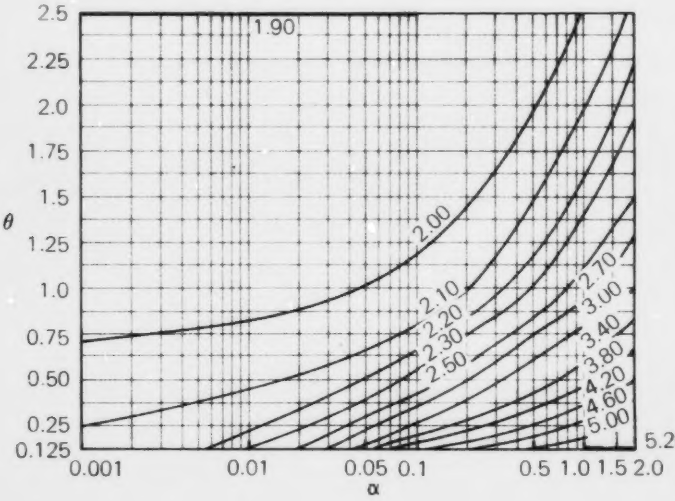
3 — 7.1.3 Transverse Bending Moments in Shallow Superstructures

3 — 7.1.3.1 Transverse Bending Moments in Slab Bridges

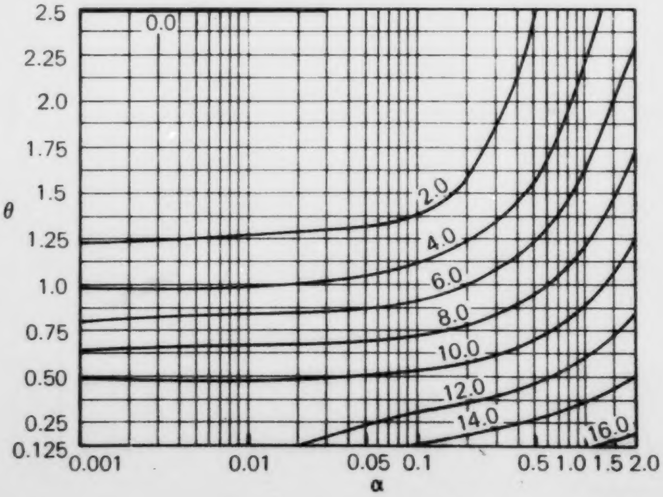
For bridges satisfying the requirements of Clause 3 — 7.1.1 the following method may be used.

a) The bridge shall be loaded by one OHBD Truck in accordance with the requirements of Section 2, so as to produce maximum longitudinal bending moment. In calculating this maximum longitudinal bending moment, the loaded span shall be treated as a simply-supported beam. The average value of M_{xave} shall then be calculated as the maximum longitudinal bending moment per unit width.

Figure **Charts for SLS I Cases in Three-Lane Bridges** **3 — 7.1.2.2(c)**



D in metres



C_f, percent

b) The design positive transverse bending moment per unit length, M_y, at mid-span and mid-width shall be calculated as follows.

i) The value of θ shall be calculated in accordance with Clause 3—7.1.2.1(e)(ii).

ii) The value of γ shall be calculated as

$$\gamma = \frac{W_c - 7.5}{9.0}$$

with a minimum of 0.0 and a maximum of 1.0, where W_c is in metres.

iii) The value of F shall be obtained from Figure 3—7.1.3.1(a) for the relevant value of θ.

iv) F_d shall be calculated from:

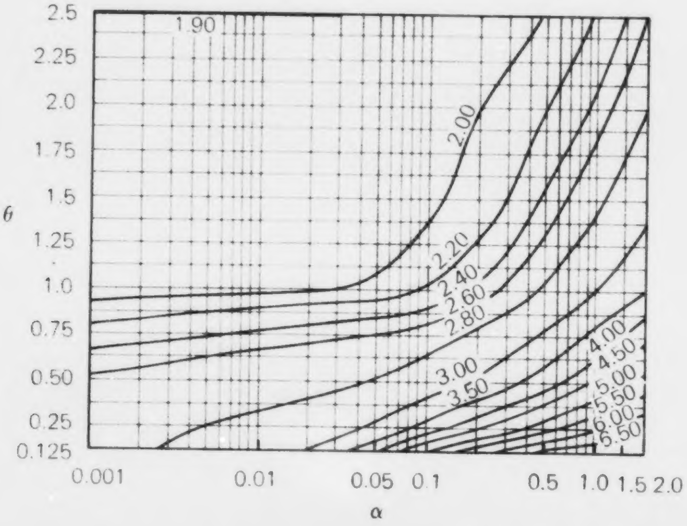
$$F_d = F(1 + 0.55\gamma).$$

v) M_y shall be calculated from

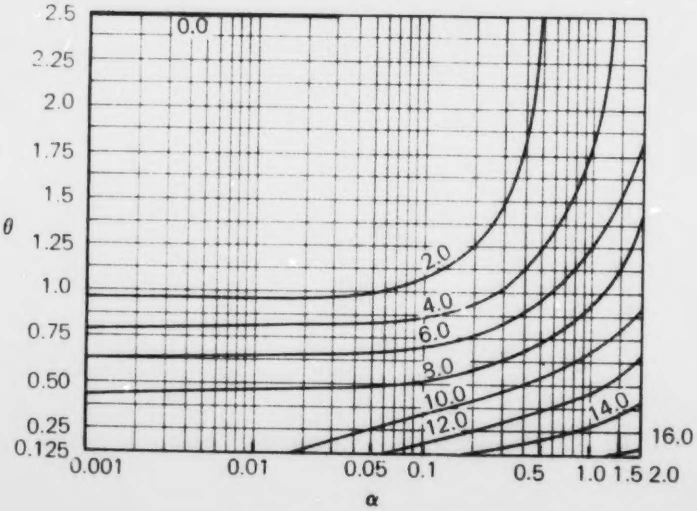
$$M_y = F_d\, M_{xave}(1 + DLA)$$

where DLA is the appropriate dynamic load allowance for a single vehicle.

Figure **Charts for SLS I Cases in Four-Lane Bridges** **3 – 7.1.2.2(d)**



D in metres



C_f, percent

c) The design positive transverse moment per unit length M_y as calculated in (b) above shall be used over the central one-half of the width of the bridge and the central two-thirds of the span. For the outer quarters of the width and the outer one-sixths of the span, the design shall be based upon 0.75 M as shown in Figure 3 – 7.1.3.1(b).

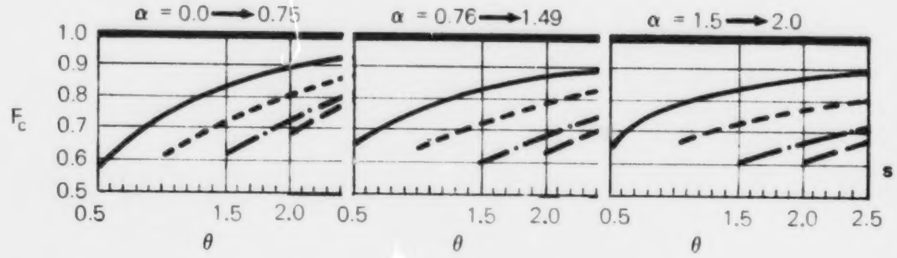
3 – 7.1.3.2 Transverse Bending Moments and Associated Longitudinal Bending Moments in Slab-On-Girder Bridges

In cases where the empirical method of Section 7 is applicable, it shall be used. Cantilever transverse bending moments in the deck slab shall be calculated in accordance with the requirements of Section 7.

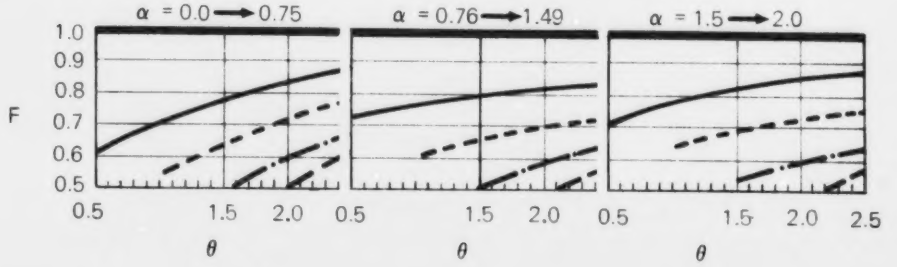
For bridges satisfying the requirements of Clause 3 – 7.1.1, in cases where the empirical method of Section 7 is not applicable, the following method may be used for obtaining live load transverse moments in the deck slabs of slab-on-girder bridges.

Figure **Charts for Correction Factor F_c Accounting for the Edge Distance of Vehicle for SLS I** **3 – 7.1.2.2(e)**

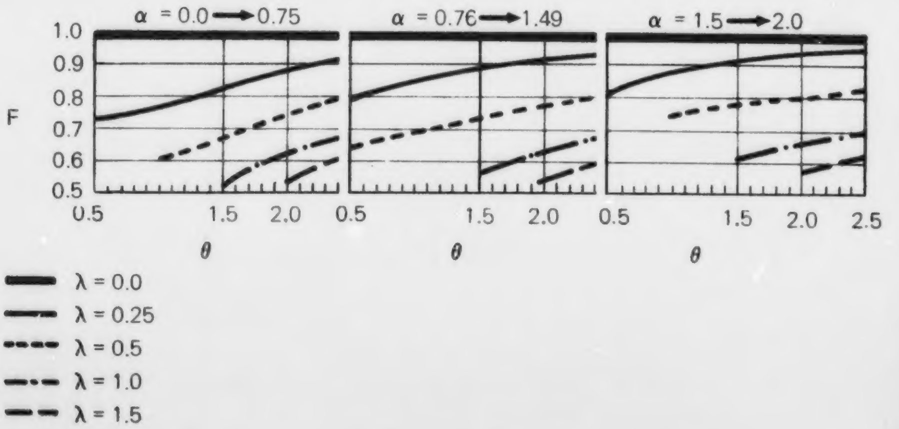
(a) 2-lane bridges



(b) 3-lane bridges



(c) 4-lane bridges



a) The governing transverse moments per unit length, M_y, shall be obtained as the sum of local and global contributions calculated in accordance with Clause 3—7.1.3.2(b) and (c) below, this sum being modified by addition of the dynamic load allowance corresponding to a single vehicle.

b) Due to local efects of the OHBD Truck, the transverse moments per unit length, M_y, shall be obtained from Figure 3—7.1.3.2(a) for external and internal panels of the cross-section corresponding to the spacing, S, of the girders. Figure 3—7.1.3.2(a) gives both negative and positive values of M_y and also gives the values, M_x, of positive longitudinal bending moment per unit width which are to be taken along with the positive values of M_y.

Figure **Distribution Factors for Transverse Bending Moments in Slab Bridges** **3—7.1.3.1(a)**

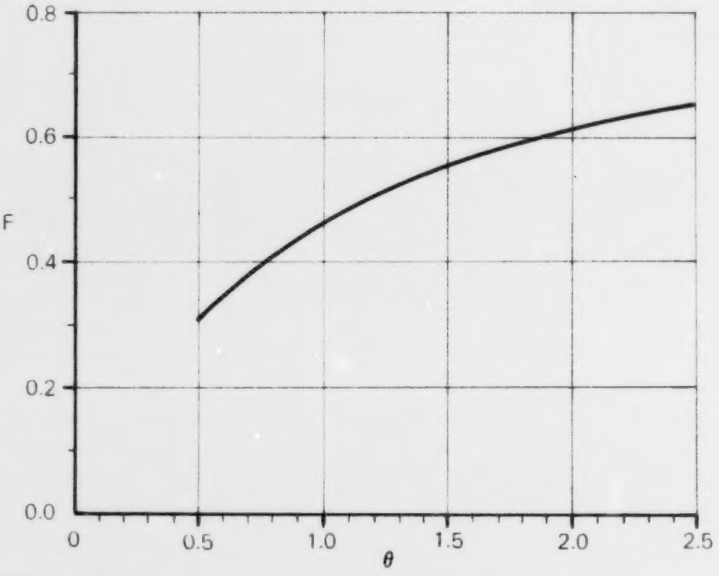
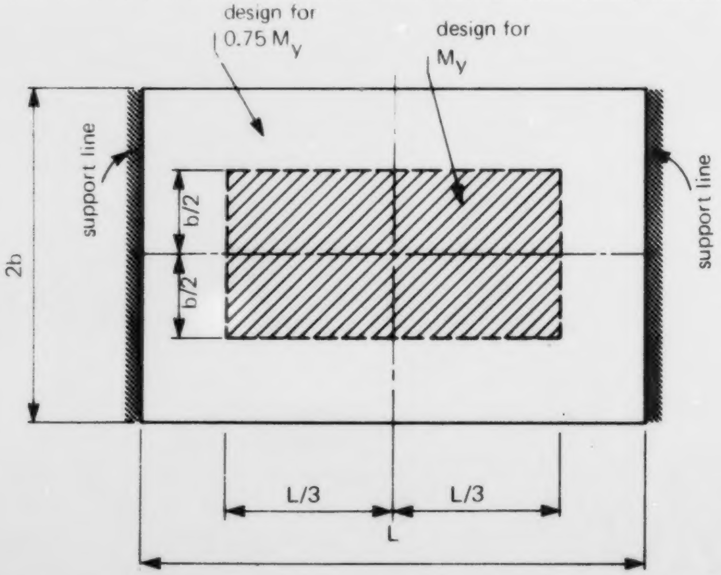


Figure **Transverse Bending Moment Zones for Design** **3—7.1.3.1(b)**



c) Due to global effects, the positive transverse moment per unit length M_y, shall be calculated from:

$$M_y = F_d M_{xave}\left[\frac{D_y}{D_x}\right]^{0.5}(1 + DLA)$$

in which F_d shall be obtained from Figure 3—7.1.3.2(b) corresponding to the value of θ obtained according to Clause 3—7.1.2.1(e)(ii), M_{xave} shall be calculated in accordance with Clause 3—7.1.3.1(a), and DLA is the appropriate dynamic load allowance for a single axle.

The value of M_y due to global effects, thus calculated, shall be used over the central one-half of the width of the bridge and the central two-thirds of the span. For the outer quarters of the width and the outer one-sixths of the span, a value of 0.75 M_y shall be taken as indicated in Figure 3—7.1.3.1(b).

Figure **Moments Due to Local Effects of OHBD Truck** **3—7.1.3.2(a)**

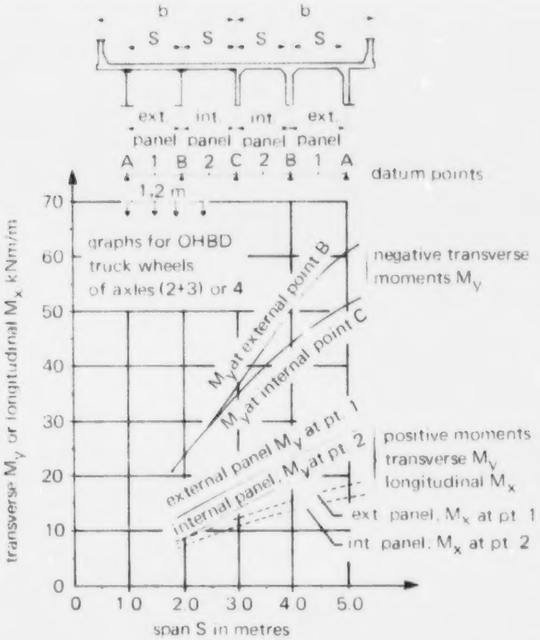
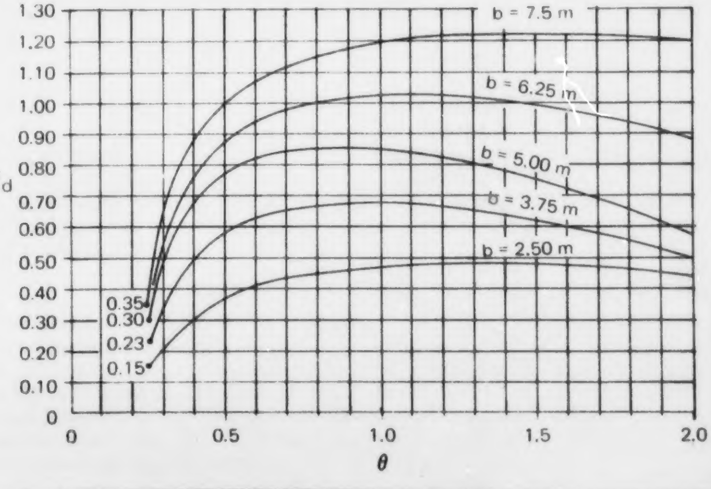


Figure **F_d Factors for Global Effects of OHBD Truck** **3—7.1.3.2(b)**



3—7.1.3.3 **Transverse Bending Moments in Transverse Wood Decking**

In cases where the superstructure comprises longitudinal girders spanned transversely by wood laminates connected only by nailing, the bending moments in the transverse wood laminates shall be calculated on the following basis:

a) An equivalent simply-supported span equal to the clear span between ends of flanges of adjacent girders plus one-half of the width of a girder flange shall be assumed.

b) One wheel load shall be assumed to be taken by a width of laminates of 0.4 m measured in the longitudinal direction.

c) If the flooring is continuous over more than two spans, the maximum bending moment shall be 0.8 times that obtained for a simple span.

3—7.1.4 **Longitudinal Shear in Shallow Superstructures**

3—7.1.4.1 **Longitudinal Shear for Ultimate Limit States and Serviceability Limit States Type II**

For design of a bridge satisfying the requirements of Clause 3—7.1.1, the following method may be used for obtaining governing live load longitudinal shear.

a) The value of D_d shall be selected from Table 3—7.1.4.1 in accordance with the type of bridge and the number of design lanes in the bridge.

b) The load fraction S/D_d shall be calculated where S is as defined in Clause 3—7.1.2.1(b).

c) Longitudinal shear shall be calculated, treating the bridge as a beam, for two load cases: the first load case comprises one line of wheels specified in Section 2, Figure 2—4.3.1.2(a) and the second load case comprises one-half of the lane load specified in Section 2, Figure 2—4.3.1.2(b). The governing longitudinal shear obtained from these two load cases shall be multiplied by the appropriate load fraction S/D_d and also by the factor (1 + DLA) to give the live load longitudinal shear for a width S of the cross-section, for both internal and external portions of the bridge. These values of the live load longitudinal shear shall be used without modification factors. Where the spacing S of longitudinal girders is less than 2.0 m, the value of D_d as obtained above shall be multiplied by

$$\left[\frac{S}{2.0}\right]^{0.25}$$

For evaluation of existing bridges satisfying the requirements of Clause 3—7.1.1, the above method may be used in conjunction with a loading comprising one-half of the relevant loading shown in Section 2, Figures 2—4.3.1.2 and 2—4.3.1.3.

Notwithstanding the above, for existing longitudinally-laminated wood bridges in which the laminates are connected only by means of nails, the value of D_d shall be taken as 0.85 m.

3—7.1.4.2 **Longitudinal Shear for Serviceability Limit States Type I**

For a bridge satisfying the requirements of Clause 3—7.1.1, live load longitudinal shear may be calculated by the method specified in Clause 3—7.1.4.1 except that the values of D_d shall be obtained from Table 3—7.1.4.2.

Table **Values of D_d in Metres for Longitudinal Shear for Ultimate Limit States and Serviceability Limit States Type II** **3—7.1.4.1**

Bridge Type		Number of Design Lanes 1	2	3	4 or more
A. Slab		2.05	1.95	1.95	2.15
B. Voided Slab		2.05	1.95	1.95	2.15
C. Slab-on-girder		1.75	1.70	1.85	1.90
H. Bridges incorporating longitudinal wood beams, but not including nail-laminated decks	(i)	1.75	1.70	1.85	1.90
	(ii)	1.85	1.90	1.95	2.00
	(iii)	1.65	1.70	1.80	1.85

Notes:

i) Refers to bridges with transverse laminated decking on sawn stringers.

ii) Refers to bridges with concrete-wood composite construction.

iii) Refers to prestressed decks.

3 — 7.1.5 **Longitudinal Bending Moments in Box Girder Bridges**

Provided that the requirements of Clause 3 — 7.1.1 are met, the simplified methods of Clauses 3 — 7.1.5.1 and 3 — 7.1.5.2 are permitted.

3 — 7.1.5.1 **Multi-Cell Bridges**

a) For a bridge satisfying the requirements of Clause 3 — 7.1.1, the simplified methods specified in Clauses 3 — 7.1.2.1 and 3 — 7.1.2.2 may be used for obtaining live load longitudinal moments for the ultimate and serviceability limit states Type II, and the serviceability limit states Type I, respectively, provided that the voids either satisfy the limitations specified in Clause 3 — 5.1.2, or are stiffened against distortion by adequate diaphragms or cross bracings. If neither of these requirements is met, the bridge can still be analysed by the methods specified in Clause 3 — 7.1.2.1 and 3 — 7.1.2.2 provided that the live load moments, thus obtained, are modified according to the following method:

i) The value of α shall be taken as 1.0

ii) The value of θ shall be taken as b/L

iii) The value of the parameter δ shall be calculated from the following equation:

$$\delta = \frac{\pi^2 b}{L^2}\left[\frac{D_x}{S_y}\right]^{0.5}$$

where S_y is the transverse shear rigidity, equal to the product of the shear modulus and the equivalent transverse shear area, s, per unit length. The equivalent transverse shear area, s, may be calculated using the formula given in Table A3 — 2(b).

b) The value of λ_m shall be obtained from the relevant chart of Figure 3 — 7.1.5.1 corresponding to the value of δ, the value of θ, and the number of design lanes in the bridge and the limit state under consideration. Thereafter, the value of D_d obtained from Clauses 3 — 7.1.2.1 and 3 — 7.1.2.2 shall be divided by λ_m in order to obtain the relevant modified D_d value, which shall be used in design.

3 — 7.1.5.2 **Multi-Spine Bridges**

a) For multi-spine bridges of concrete construction or of steel construction with at least three spines, and having a cross-section in which the voids defined by the spines either

- satisfy the limitations of Clause 3 — 5.1.2, or
- are stiffened against distortion by adequate diaphragms or cross bracings,

the simplified methods of Clause 3 — 7.1.2 may be used, provided that the dimension S is taken to be centre-to-centre of spines and the values of α and θ shall be taken as follows:

$$\alpha = 2.0$$

$$\theta = \frac{2b}{L}\left[\frac{D_x}{D_{xy}}\right]^{0.5}$$

Table **Values of D_d in Metres for Longitudinal Shear for Serviceability Limit States Type I** **3 — 7.1.4.2**

Bridge Type		Number of Design Lanes 1	2	3	4 or more
A. Slab		2.05	2.10	2.25	2.50
B. Voided Slab		2.05	2.10	2.25	2.50
C. Slab-on-girder		1.75	1.80	1.90	2.10
K. Bridges incorporating longitudinal wood beams, but not including nail-laminated decks	(i) (ii) (iii)	1.75 1.85 1.65	1.80 1.95 1.75	1.90 2.10 1.85	2.10 2.30 2.00

Notes:

i) **Refers to bridges with transverse laminated decking on sawn stringers.**

ii) **Refers to bridges with concrete-wood composite construction.**

iii) **Refers to prestressed decks.**

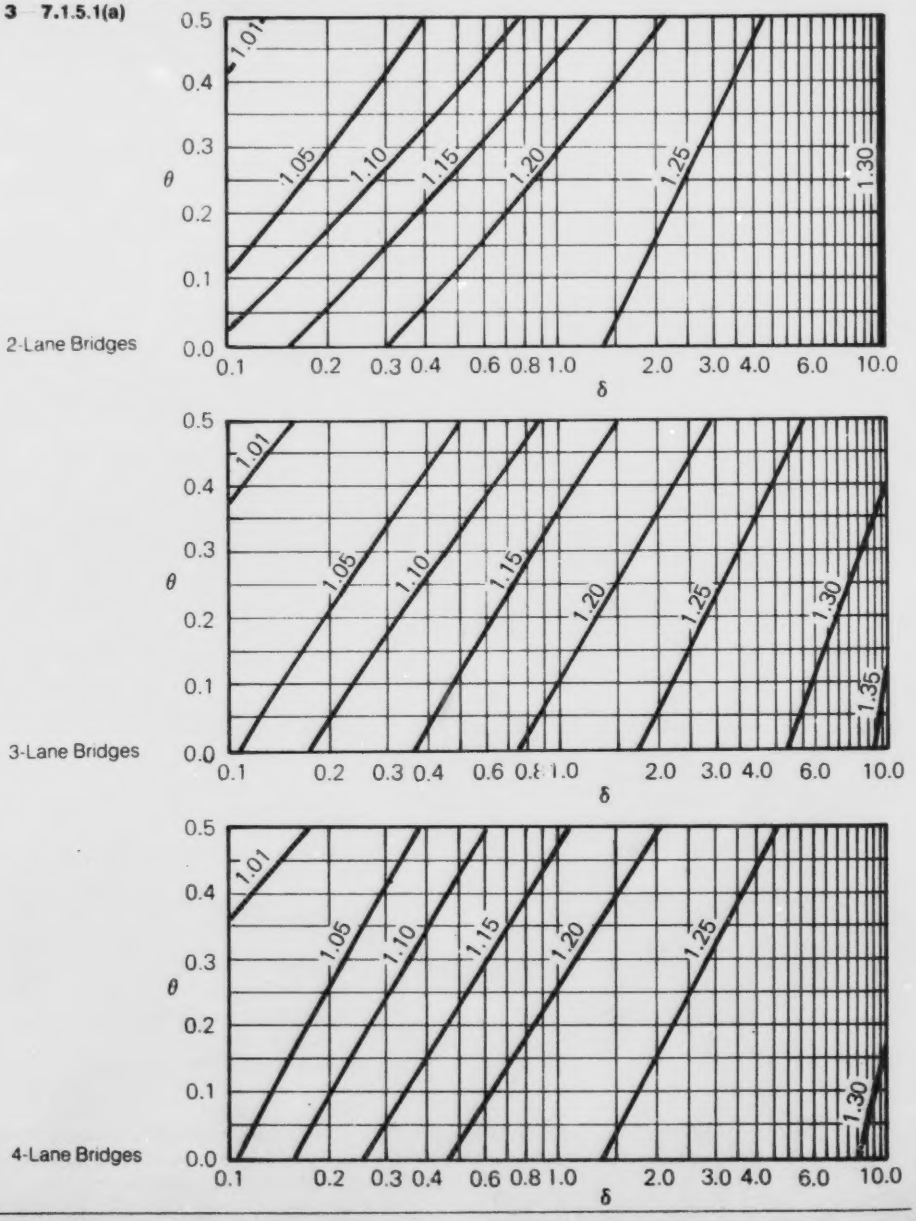
b) For multi-spine bridges of steel-concrete-composite construction, for ultimate limit state and serviceability limit states Type II, the live load bending moment for each spine shall be determined by applying to the spine the fraction L_L of each of the loads specified in Clause 3—7.1.2.1(c) separately and the more severe case adopted, where L_L is given by the following equation:

$$L_L = 0.95\,W_L$$

where: $W_L = 0.1 + 1.7H + \dfrac{0.85}{N_w}$

where: N_w = number of design lanes

Figure λ$_m$ Charts for SLS I 3—7.1.5.1(a)



$$H = \frac{N_w}{\text{number of spines}}$$

The value of H shall not be less than 0.5 nor greater than 1.5. For the serviceability limit state Type I the fraction L_L shall be determined by the equation:

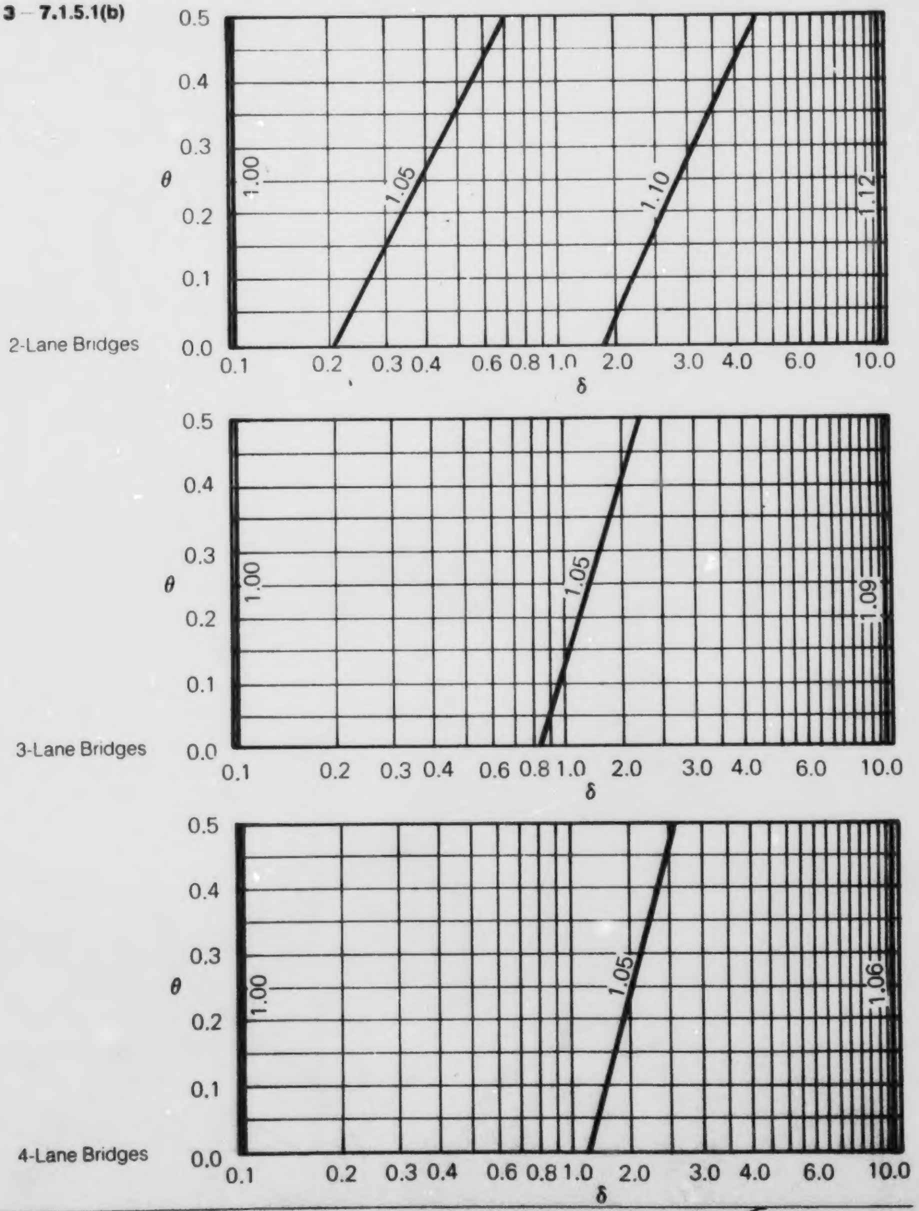
$$L_L = 0.55W_L$$

The appropriate dynamic load allowance (DLA) shall be used to obtain the governing live load moments.

3 — 7.1.6 Longitudinal Shear in Box Girder Bridges

Provided that the requirements of Clause 3 — 7.1.1 are met, the simplified method of Clause 3 — 7.1.6.1 is permitted.

Figure **λ_m Charts for ULS and SLS II** **3 — 7.1.5.1(b)**



3—7.1.6.1 **Multi-Cell Bridges**

For bridges having a cellular cross-section in which the voids either:

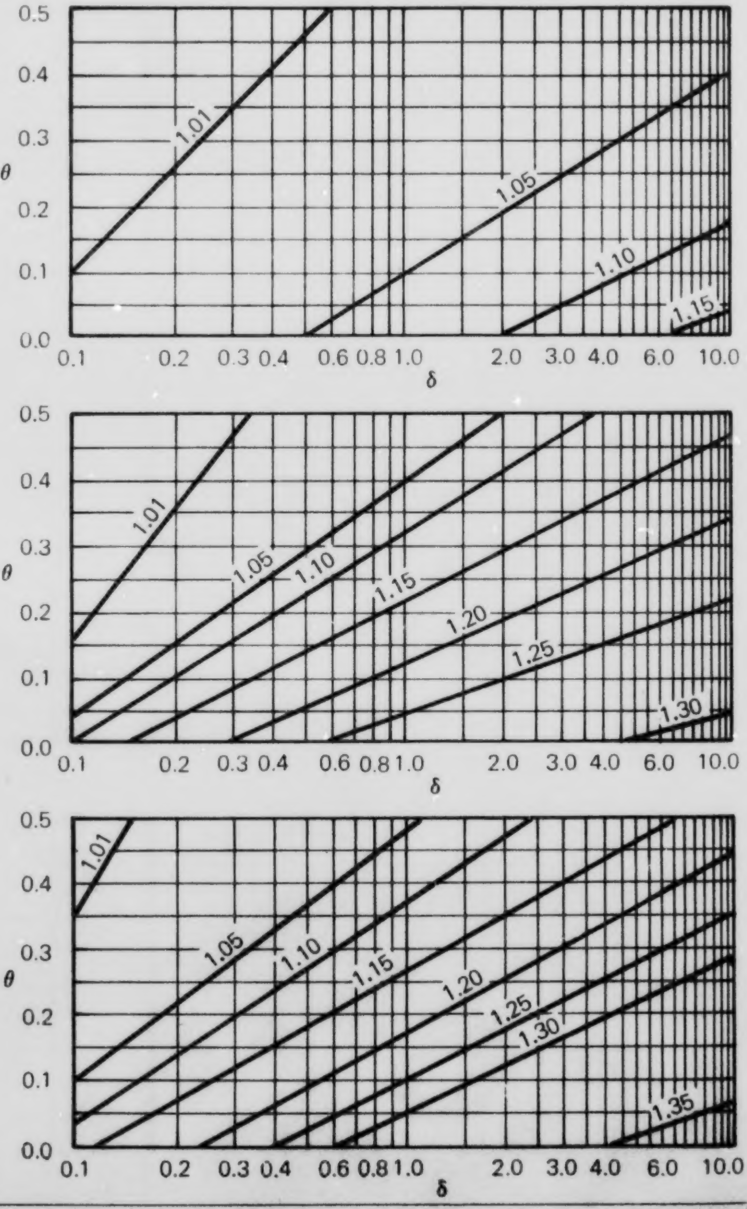
- satisfy the limitations of Clause 3—5.1.2, or
- are stiffened against distortion by adequate diaphragms or cross bracings,

the simplified methods of Clause 3—7.1.4 may be used, provided that the following modifications are employed.

i) The value of α shall be taken to be 1.0.
ii) The value of θ shall be taken to be b/L.
iii) The value of an additional parameter, δ, shall be calculated from:

$$\delta = \frac{\pi^2 b}{L^2}\left[\frac{D_x}{S_y}\right]^{0.5}$$

Figure **λ_s Charts for SLS I** **3—7.1.6.1(a)**



2-Lane Bridges

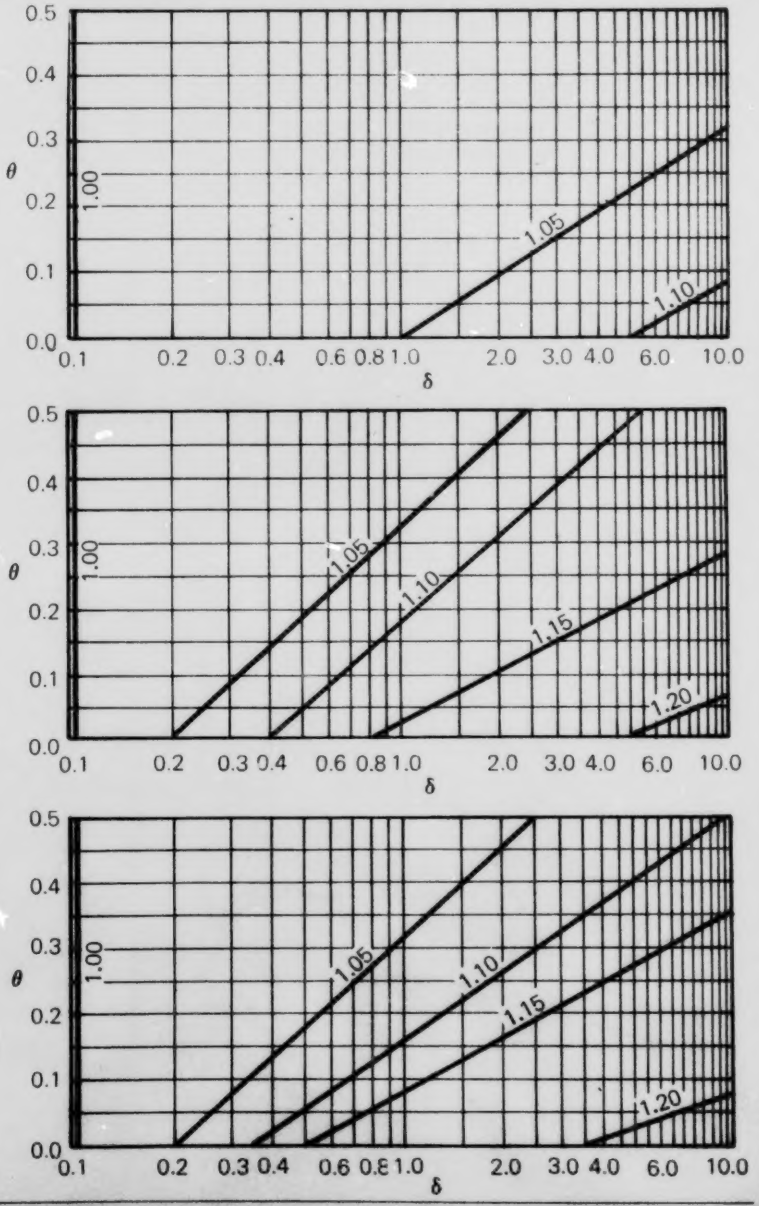
3-Lane Bridges

4-Lane Bridges

where S_y is the transverse shear rigidity and is the product of the shear modulus and the equivalent transverse shear area per unit length. The equivalent transverse shear area may be calculated using the formula given in Table A3 — 2(b).

iv) For each loading case, the value of D_d to be used for the calculation of longitudinal shears in cellular cross-section bridges shall be $1.0/\lambda_s$ times the value of D_d obtained from Clause 3 — 7.1.4 for slab bridges. The value of λ_s shall be obtained from the relevant chart of Figure 3 — 7.1.6.1(a) for SLS I and Figure 3 — 7.1.6.1(b) for ULS and SLS II, using values of θ and δ determined in accordance with Clause 3 — 7.1.5.1

Figure **λ_s Charts for ULS and SLS II** **3 — 7.1.6.1(b)**



3—7.1.6.2 **Multi-Spine Bridges**

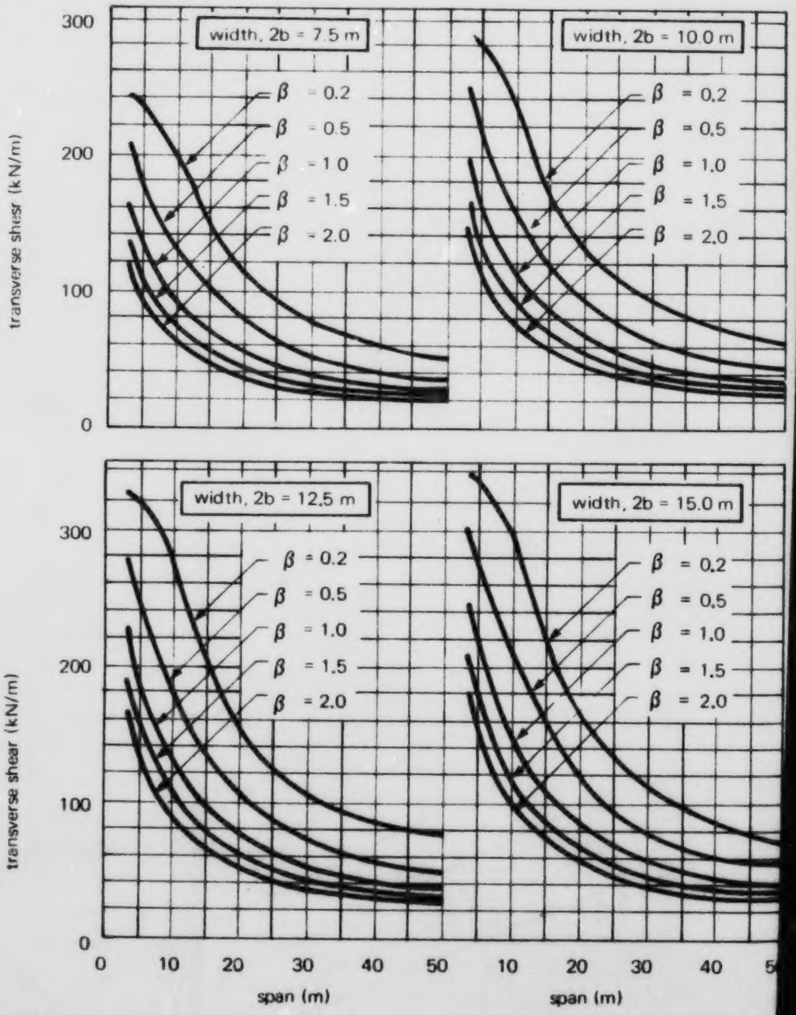
a) For multi-spine bridges of concrete construction or of steel construction with at least three spines, and having a cross-section which satisfies the requirements of Clause 3—7.1.5.2(a), the value of D_d for longitudinal shear shall be 0.9 times the value obtained in accordance with Clause 3—7.1.5.2(a) for longitudinal moment. Thereafter, the method of Clause 3—7.1.4.1(c) shall be used for the loading relevant to the limit state under consideration.

b) For multi-spine bridges of steel-concrete composite construction, the method of Clause 3—7.1.5.2(b) may be used, provided that the load fraction L_L is determined by the following equations:

$L_L = 1.05W_L$, for ultimate limit state and serviceability limit states Type II;

$L_L = 0.60W_L$, for serviceability limit states Type I.

Figure **Charts for Transverse Shear for Shear-Connected Beam Bridges** **3—7.1.7(a)**



3—7.1.7 Transverse Shear in Shear-Connected Beam Bridges and Prestressed Wood Decks

The maximum intensity of transverse shear shall be assumed to occur when there is only one design vehicle on the bridge.

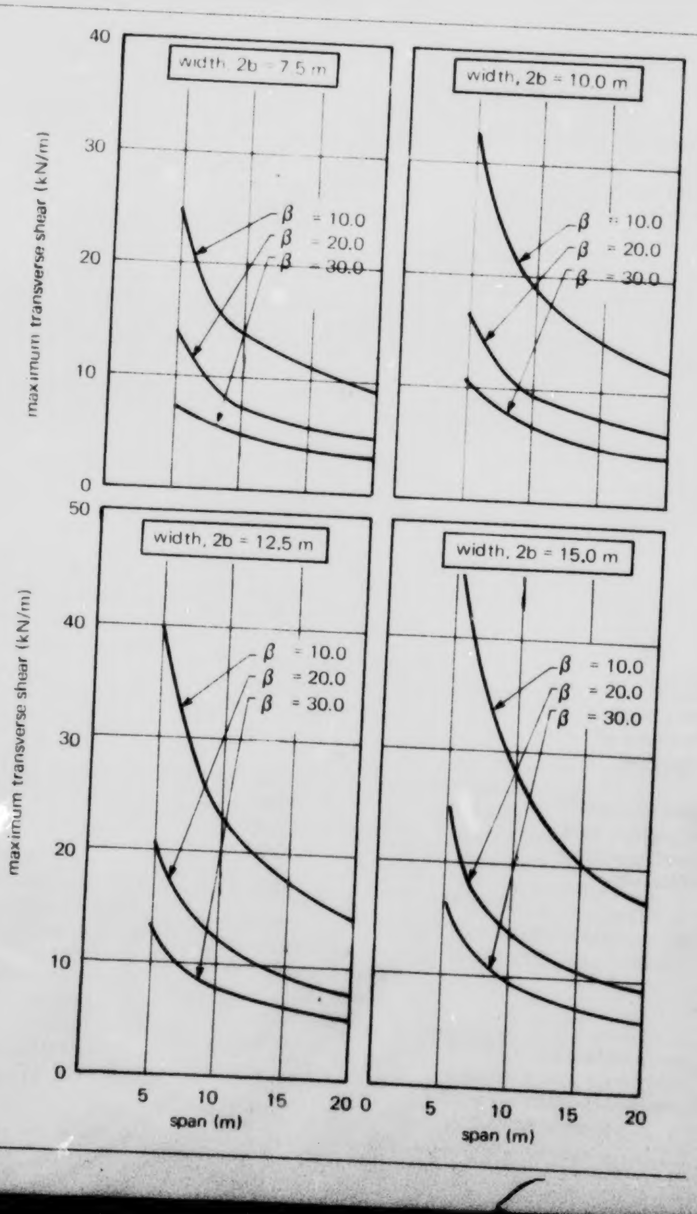
The following simplified method may be used:

i) Calculate $\beta = \pi \left[\frac{2b}{L}\right] \left[\frac{D_x}{D_{xy}}\right]^{0.5}$

ii) Using the charts of Figures 3—7.1.7(a) for shear-connected beam bridges and 3—7.1.7(b) for prestressed wood decks, for the appropriate values of span and β, obtain the value of transverse shear intensity. Use linear interpolation for this intensity in the case of a width falling between the widths given on the charts.

iii) Use the appropriate dynamic load allowance for single vehicles to obtain the design intensity of transverse shear.

Figure **Charts for Transverse Shear for Prestressed Wood Decks** **3—7.1.7(b)**



3—7.1.8 **Analysis of Transverse Beams**

In analysing transverse beams covered by a concrete deck, the effect of a concentrated load situated between two parallel adjacent transverse beams may be calculated as follows provided that there are no stringers or, if stringers are present, the decking is continuous over the stringers.

i) The original concentrated load shall be replaced by statically equivalent concentrated loads on the transverse beams, acting at points on the longitudinal line through the original concentrated load.

ii) Each equivalent concentrated load shall itself be replaced by a distributed load, with triangular load intensity, extending symmetrically on either side of the concentrated load position over a length of the transverse beam which corresponds to a dispersion angle from the original concentrated load's position of not more than 30° on either side of the longitudinal direction.

3—7.1.9 **Analysis of Stringers in Truss and Arch Bridges**

In analysing the stringers of truss and arch bridges, the portion of the bridge between adjacent floor beams may be analysed by the methods of Clauses 3—6.1.1 and 3—6.1.2 for dead load and 3—7.1.2, 3—7.1.3 and 3—7.1.4 for live load.

Refined Methods of Analysis 3—7.2

In cases where the simplified methods of Clause 3—7.1 are not applicable, a refined method in accordance with Clause 3—8 shall be employed.

Permitted Refined Methods of Analysis 3—8

General Applications 3—8.1

General methods which are approved for the analysis of the appropriate bridge types, subject to the provisions of Clause 3—9, are as follows:

i) grillage analogy methods
ii) orthotropic plate theory methods
iii) finite element methods
iv) finite strip methods
v) folded plate methods

Specific Applications 3—8.2

Influence surfaces may be used to evaluate relevant responses in bridge superstructures provided that they are developed from the refined methods listed in Clause 3—8.1 or from model analysis in accordance with Clause 3—8.3. The use of influence surfaces developed by other methods, and the use of other methods, requires approval.

Model Analysis 3—8.3

The use of model analysis, involving the testing of a physical model of the whole or part of the bridge, is acceptable as an alternative to, or in addition to, other methods of analysis permitted in this section. The model analysis, and the interpretation of the results for the purpose of design, shall be subject to approval and shall be carried out by a person or persons qualified in this field. The analysis shall be such that the resulting design ensures a level of safety and performance at least equivalent to that provided for, or implicit in, other methods of analysis.

Selection of Methods of Analysis 3—9

Unless specified elsewhere in the Code or otherwise approved, the method or methods of analysis shall be selected from Table 3—9.

Idealization of Structure and Interpretation of Results 3—10

General 3—10.1

Prior to analysis, the structure, the boundary conditions, and the loading shall be idealized in such a way that the total idealization represents realistically the properties of the actual structure, the actual boundary conditions, and the actual dead and applied loads.

Idealization of structure in accordance with Appendix A3—2 is permitted. In applying the results of the analysis to the actual structure, the structural responses carried by any given component of the mathematical model shall be deemed to be carried by the portion or portions of the actual structure for which the given component is the analogue.

Table **Selection of Methods of Analysis** **3—9**

Method of Analysis	Bridge Types listed in Clause 3—1 for which the Method is Permitted	Limitations of Applicability
Simplified Method of Clause 3—6.1.2 for Dead Load	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss Arch Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder—Single Cell Box Girder—Multi-Cell Box Girder—Multi-Spine	Elements of structure must meet requirements of Clause 3—6.1.2.
Simplified Methods of Clauses 3—7.1.2 3—7.1.3 3—7.1.4 3—7.1.7 for Live Load	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss, Arch, Rigid Frame, Bridges incorporating Wood Beams	Structure must meet requirements of Clause 3—7.1.1.
Simplified Methods of Clauses 3—7.1.5 3—7.1.6 for Live Load	Box Girder—Multi-Cell Box Girder—Multi-Spine	Structure must meet requirements of Clause 3—7.1.1, and Clauses 3—7.1.5.1, 3—7.1.5.2, 3—7.1.6.1, as applicable.
Grillage Analogy Method	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss Arch Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder—Multi-Cell Box Girder—Multi-Spine	Not applicable to voided slab and box girder bridges in which the number of cells is less than three.
Orthotropic Plate Theory Methods	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss Arch Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder—Multi-Spine	Structure must meet requirements of Clause 3—7.1.1 and also, i) edge stiffening, if present, to be idealized as specified in Appendix A3—1, and ii) multi-spine bridges to be of concrete construction and have four or more spines.
Finite Element Method	All bridge types	For shear-connected beam bridges, special elements, having zero transverse rigidity, may be required.

Table **Selection of Methods of Analysis (cont'd)** **3—9**

Method of Analysis	Bridge Types listed in Clause 3—1 for which the Method is Permitted	Limitations of Applicability
Finite Strip and Folded Plate Methods	Slab Voided Slab Slab-on-Girders Shear-Connected Beams Floor Systems of Truss Arch Rigid Frame Bridges incorporating Longitudinal Wood Beams Box Girder — Single Cell Box Girder — Multi-Cell Box Girder — Multi-Spine	Not applicable to bridges with any of the following: i) skew angle greater than permitted by Clause 3—6.1.1; ii) longitudinal variations in transverse directions; iii) support conditions other than those permitted by Clause 3—6.1.1 iv) intermediate diaphragms. For shear-connected beam bridges, special elements, having zero transverse rigidity, may be required
Conventional Methods of Analysis for: Trusses Arches Rigid Frames	 Truss Arch Rigid Frame	
Influence Surfaces	All bridge types	In accordance with Clause 3—8.2.1
Model Analysis	All bridge types	In accordance with Clause 3—8.3
Other Methods	All bridge types	Require approval.

Effective Flange Widths for Bending **3—10.2**

In calculating bending resistances and bending stresses in slab-on-girder bridges and box girder bridges having a concrete slab, a reduced cross-section shall be used comprising a left-hand overhang, a central portion and a right-hand overhang. The overhang B_e shall be determined as follows.

a) For simply-supported single-span bridges, the ratio B_e/B shall be related to the ratio L/B, by the expression:

$$\frac{B_e}{B} = 1 - \left(1 - \frac{L}{15B}\right)^3, \not> 1.0$$

where: L = span,

B_e = the dimension shown in Figure 3—10.2(a) for various types of bridge cross-section,

B = the dimension shown in Figure 3—10.2(a) for various types of bridge cross-section,

and both B_e and B are measured from the face of the web or haunch.

b) For continuous-span bridges, and for bridges having moment restraint at an end support, the bridge length shall be divided into alternating regions of positive moment and negative moment due to dead load. The lengths of these regions shall be used as equivalents for the span L and the effective flange widths B_e of each region, in turn, shall be found from the expression given above. A step-wise change of effective flange width from one region to the next shall be assumed.

For the purpose of determining effective flange width only, in a bridge that is free of moment restraint at its ends, the positive moment region of an end span shall be taken as the outermost 80% of it, and the positive moment region of an interior span shall be taken as the central 60% of it, in which case the regions of positive and negative moment take the pattern shown in Figure 3—10.2(b).

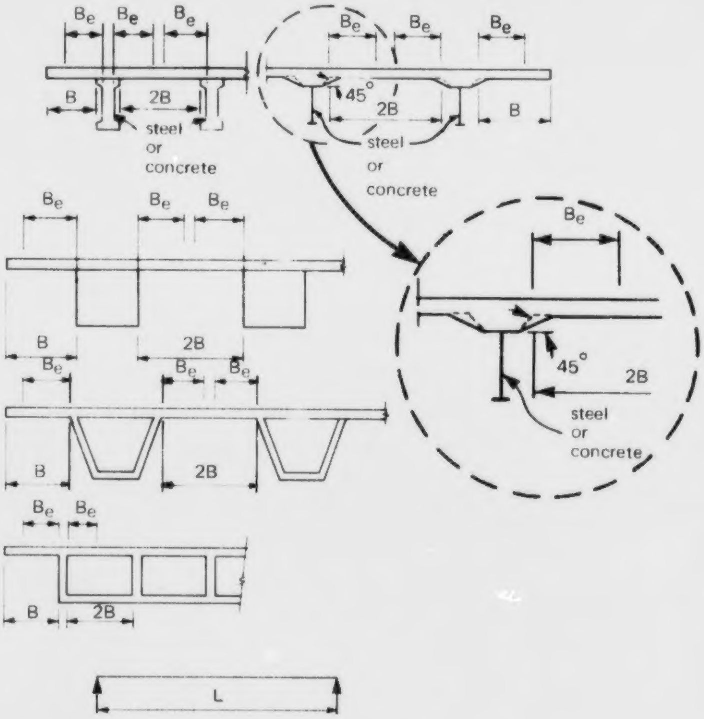
Idealization for Analysis 3—10.3

For slab-on-girder bridges, the section properties for the purposes of analysis shall be those of the uncracked section. The section shall be treated as composite or non-composite according to the presence or absence of shear connectors.

Idealization for Calculating Stresses in Negative Moment Regions 3—10.4

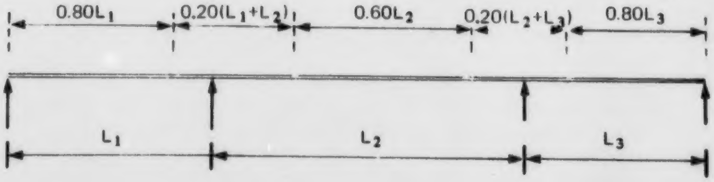
For composite slab-on girder bridges, the section properties in the negative moment region, for the purposes of proportioning of members, shall be in accordance with the provisions of Sections 8, 9, 10 and 13.

Figure **B_e and B for Various Cross-Sections** **3—10.2(a)**



igure **Positive and Negative Moment Regions for the Purpose of Determining Effective Flange Width** **3—10.2(b)**

Effective Flange Width Points of Contraflexure Shown



Appendix **Factors Affecting Structural Response** **A3 — 1**

Factors affecting structural response are given in this Appendix. The method of analysis chosen shall be capable of evaluating this affected behaviour.

a) **Continuity of Spans**

When the simplified methods given in Clauses 3 — 7.1.2, 3 — 7.1.4, 3 — 7.1.5 and 3 — 7.1.6 are to be used for a multi-span bridge, the effect of continuity may be accounted for in the following way.

Provided that the ends of the bridge are free of externally applied restraint against rotation, the value of L in the formula for calculating θ shall be taken as:

- i) for positive moments in an exterior span, 80% of the distance between the external support and the internal support;
- ii) for positive moments in an interior span, 60% of the distance between the internal supports;
- iii) for negative moments in the region of an internal support, 20% of the sum of the spans on either side of the support.

The positive moment and negative moment regions, for the purposes of establishing the value of L in the formula for θ only, are then as shown in Figure A3 — 1(a). Points of inflexion shall not be assumed to occur at the positions shown in Figure A3 — 1(a) for any purpose other than establishing the value of L in the formula for θ, except as provided for in Clause 3 — 10.2.

Figure **Assumed Dead Load Points of Contraflexure** **A3 — 1(a)**

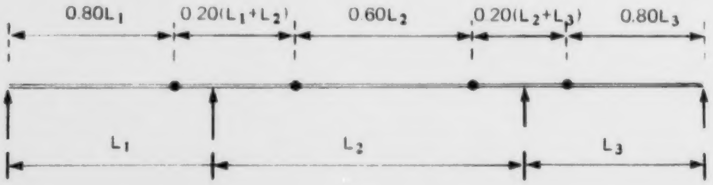


Figure **Bridge Curved in Plan** **A3 — 1(b)**

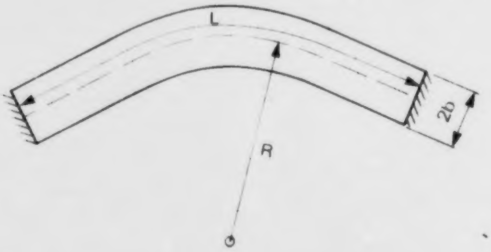
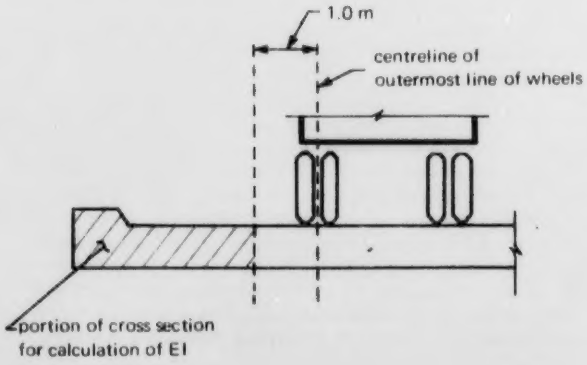


Figure **Portion of Cross-Section for Edge Stiffening Analysis** **A3 — 1(c)**


1.0 m
centreline of outermost line of wheels
portion of cross section for calculation of EI

b) Plan Geometry

i) In the case of shallow superstructures on skew spans, the following applies.

- For skew angles not exceeding 20°, the angle of skew may be ignored provided that the analysis is carried out for a right span which is equal to the skew span of the actual structure.
- For skew angles greater than 20°, the effects of longitudinal and transverse torsion under live and dead loads and pretressing forces must be accounted for.

ii) Bridges Curved in Plan

For both live loads and dead loads, longitudinal twisting moments and the associated effects of torsional and distortional warping must be adequately assessed by the method of analysis used.

If L^2/bR for a bridge as shown in Figure A3 — 1(b) is not greater than 1.0, the bridge may be treated as straight.

If R exceeds 90 m, then, for the purpose of calculating longitudinal bending moments, the bridge may be treated as straight.

iii) Other Plan Geometries

When a bridge superstructure of plan geometry other than rectangular, skew or curved is analysed, the method of analysis must be capable of deriving all relevant structural responses and be compatible with the requirements given in (i) and (ii) for the analysis of similar types of bridges.

c) Edge Stiffening

For right single span bridge superstructures incorporating slabs or slabs-on-girders or voided slabs, the following method may be used to account for the effects of edge stiffening.

i) For longitudinal bending moments for Ultimate Limit State and Serviceability Limit States Type II, the method of Clause 3 — 7.1.2.1 shall be used for calculating longitudinal bending moments, except that Clause 3 — 7.1.2.1(e) (iv) shall be replaced by:

The value of λ shall be calculated from:

$$\lambda = \left[\frac{EI}{LD_x}\right]\left[\frac{D_x}{D_y}\right]^{0.25}$$

where EI is the flexural rigidity of that part of the cross-section, on one side of the bridge and including the edge stiffening, which lies outside of a point 1.0 m outer of the centreline of the outside line of wheels, as shown in Figure A3 — 1(c).

Thereafter, the values of a correction factor F_c shall be obtained from the relevant chart of Figure A3 — 1(d) for internal and external portions. The final values of D_d shall be obtained from:

$$D_d = \frac{D}{F_c}\left[1 + \frac{\mu C_f}{100}\right]$$

for internal and external portions.

ii) For longitudinal bending moments for Serviceability Limit States Type I, the method of Clause 3 — 7.1.2.2 shall be used for calculating longitudinal bending moments. In calculating λ, the value of EI shall be calculated as the flexural rigidity of that part of the cross-section, on one side of the bridge and including the edge stiffening, which lies outside of a point 1.0 m outer of the centreline of the outside line of wheels, as shown in Figure A3 — 1(c).

d) Longitudinal Variation of Transverse Cross-Section

The analysis of a bridge superstructure incorporating varying transverse cross-section (introduced, for example, by haunches over supports in slab or slab-on-girder bridges) shall take account of the varying cross-section where such variation would influence the longitudinal and transverse distribution characteristics of the bridge.

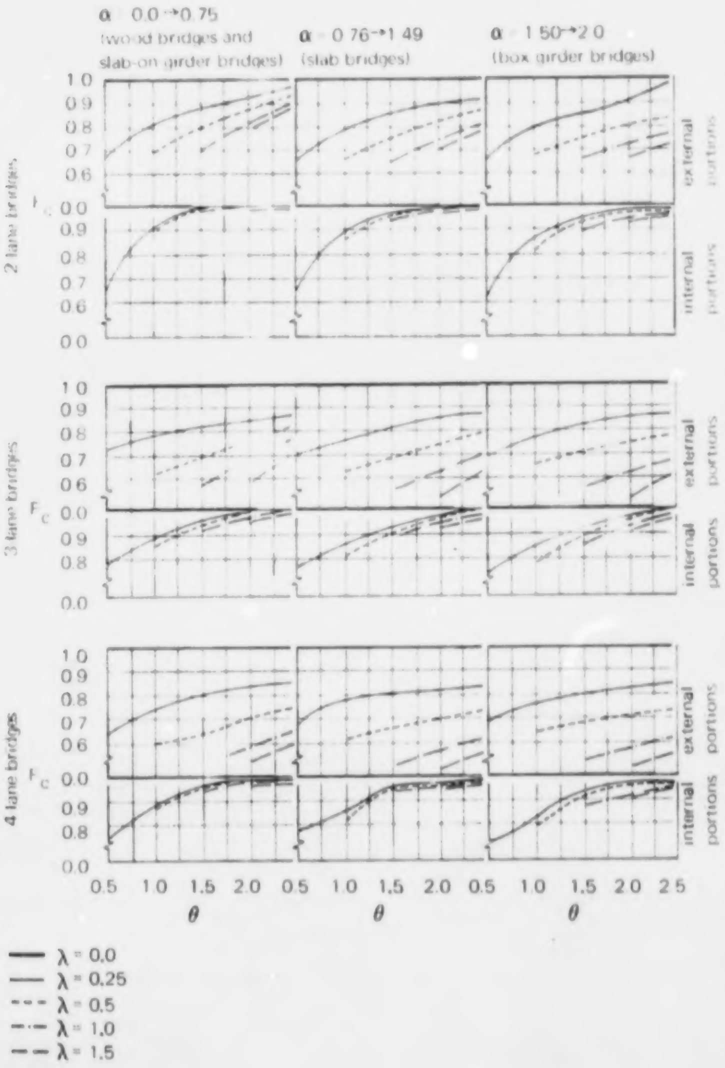
In the absence of a more accurate analysis, the following approximations may be used in conjunction with the method of Clause 3 — 7.1.2. For a slab-on-girder bridge with girders having variable moment of inertia as shown in Figure A3 — 1(e), in the negative moment region replace the haunched girder by one of constant moment of inertia in such a way that the two girders would have the same tip deflection when treated as cantilevers between the support and the point of contraflexure, and loaded with a point load at the tip. This applies in the longitudinal direction; no adjustment is required in the transverse direction.

e) **Transverse Variation of Longitudinal Cross-Section**
A method of analysis which assumes that all longitudinal cross-sections of the bridge are equal may be used for the analysis of bridges having slabs which are tapered in the vicinity of their free edges, provided that the length of the taper in the transverse direction does not exceed 2.5 m, as shown in Figure A3 — 1(f).
The tapered portion shall be replaced by an equivalent rectangular portion of the same cross-sectional area and of the full depth of slab.

f) **Diaphragms and Cross Frames**
For shallow superstructure type bridges, the effect of diaphragms and cross frames between supports on the structural responses may be ignored. In the case of box girders, all diaphragms and cross frames shall be taken into consideration.

g) **Wind Bracing**
Engineering judgement shall be used to decide whether or not a wind bracing

Figure **Correction Factors F_c Corresponding to ULS and SLS II Cases** **A3 — 1(d)**



will act integrally with the rest of the structure; if it does, the analysis used shall be able to predict the forces in the bracing.

h) **Interaction of Floor System and its Support System**
In truss bridges and arch bridges where the floor system is connected to the truss or arch in such a way that at least a part of the floor system acts integrally with the trusses or arches, the effective contribution from the floor system may be included by means of an approved method of analysis.

i) **Barrier and Parapet Walls**
In cases where the bridge incorporates barrier or parapet walls which are structurally integral with the bridge structure, the distribution of loads shall be calculated on the following basis:

- For Serviceability Limit States Type I, the barrier or parapet walls shall be included in the bridge cross-section and the effect of such walls shall be included in accordance with the provisions of Appendix Clause A3 — 1(c).
- For Serviceability Limit States Type II and Ultimate Limit States, the effect of barrier and parapet walls shall be ignored.

Notwithstanding the above, the beneficial effect of barrier walls, on the strength of deck slabs only, must always be included in the analysis.

Figure **Idealization of Section of Varying Moment of Inertia by a Constant Moment of Inertia Section** **A3 — 1(e)**

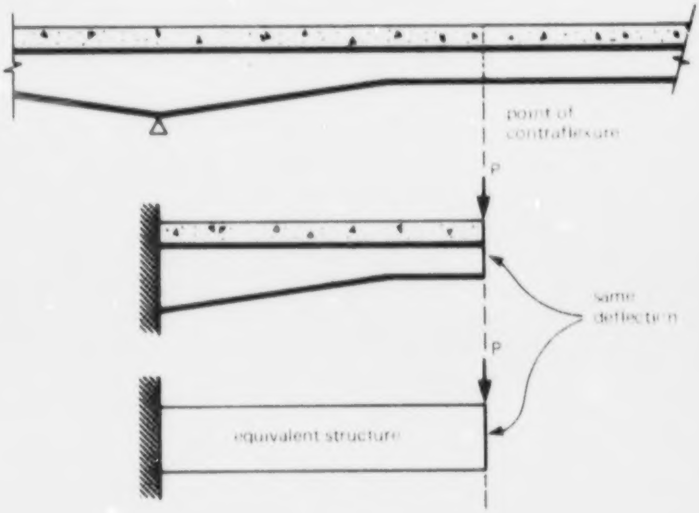
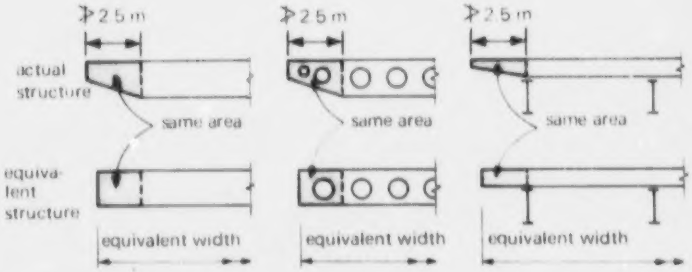

point of contraflexure
P
same deflection
P
equivalent structure

Figure **Idealization of Cross-Section with Varying Thickness** **A3 — 1(f)**


≯2.5 m
≯2.5 m
≯2.5 m
actual structure
same area
same area
same area
equivalent structure
equivalent width
equivalent width
equivalent width

j) **Support Conditions Other Than Line Support**
In cases where the support condition includes isolated supports or other circumstances leading to an irregular pattern of support forces, the method of analysis used shall be capable of assessing the local behaviour arising from the support condition.

k) **Movement of Supports;**
Also Supports for Continuous and Skew Spans
The methods of analysis shall take into account the anticipated support conditions in new bridges and the actual support conditions in existing bridges, either directly or through subsequent adjustments to the results of the analysis. In continuous or skew structures, the analysis shall be capable of taking into account differential settlement.

Since, in skew span bridges the reactions over supports do not follow the same pattern as their right span counterparts, and the reaction at bearings in obtuse-angled corners of skew slab bridges can be several times larger than those in comparable rectangular bridges, the analysis of support reactions shall be based upon a realistic modelling of the support conditions.

l) **Temperature Effects**
Stresses due to change in the mean temperature of the bridge, or to temperature gradients, shall be assessed in accordance with the provisions of Section 2.

m) **Creep and Shrinkage**
The structural response of a bridge superstructure due to creep and shrinkage effects of concrete shall be provided for in accordance with the provisions of Sections 2 and 9.

n) **and o) Secondary Moments and Elastic Shortening**
The effects of secondary moments and elastic shortening shall be taken into account and elastic methods of analysis shall be used for this purpose.

p) **Construction Sequence**
Due account shall be taken of the change in nature of the structural system and in material properties which occurs during the construction sequence. The behaviour at any one point in time during the construction sequence shall be analysed by elastic methods of analysis.

Appendix **Idealization of Structures** **A3 — 2**

a) **Two-Dimensional Analysis of Shallow Steel or Concrete Superstructures**
The bridge types referred to are slabs, slab-on-girders, voided slabs, and shear-connected beams. In considering the flexural behaviour of such bridge types by two-dimensional mathematical models the following parameters are required:

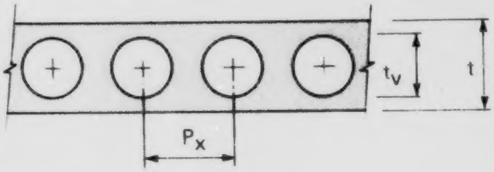
i) *Parameters depending upon the material:*
E = modulus of elasticity
ν = Poisson's ratio, to be taken as 0.15 for concrete and 0.30 for steel
G = Shear modulus = $E/2(1+\nu)$
n = modular ratio, E_s/E_c

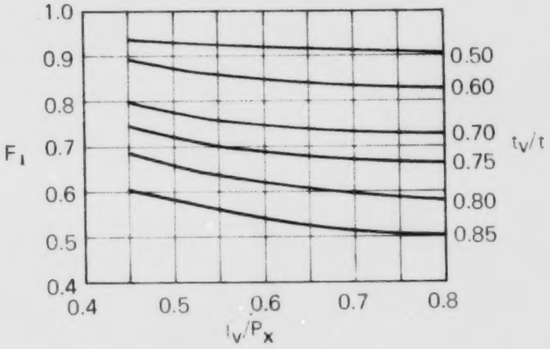
ii) *Parameters depending upon the cross-section*
i = longitudinal moment of inertia per unit width
i_o = longitudinal torsional inertia per unit width
j = transverse moment of inertia per unit length
j_o = transverse torsional inertia per unit length
s = shear area per unit length, this being required only for slabs with rectangular voids.

Structures of the above types may be idealized either as grillages or as orthotropic plates. The properties required for the two idealizations shall be calculated as shown in Table A3 — 2(a) and the values of the parameters shall be calculated as shown in Table A3 — 2(b). For both reinforced concrete and prestressed concrete, the uncracked section shall be used in calculating these parameters. The factors F_1 and F_2 indicated in Table A3 — 2(b) for voided slabs shall be obtained from Figure A3 — 2(a). In other cases, the torsional inertia of a section may be calculated by dividing the section into a number of rectangles and adding the torsional inertias of all rectangles. The torsional inertia of a single rectangle having sides a and b shall be taken to be $J = Kab^3$ where a is the longer and b the shorter of the two sides of the rectangle and K is a constant depending upon the ratio of a/b, which can be interpolated from Table A3 — 2(c).

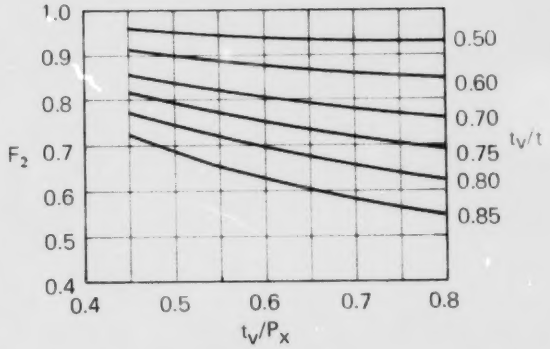
In the absence of a more detailed analysis, the equivalent areas for in-plane analysis of slabs with circular voids shall be taken as follows:

Figure **Modification Factors for Voided Slabs** **A3 — 2(a)**



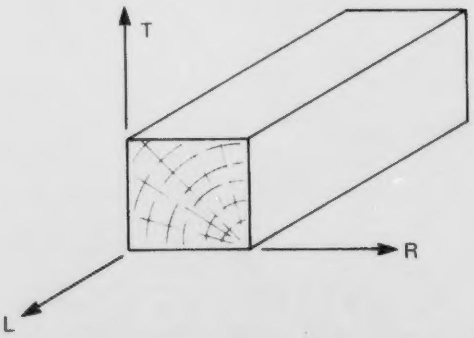


coefficients for D_y



coefficients for D_{xy}

Figure **Principal Directions in a Wood Specimen** **A3 — 2(b)**



- A_x, the equivalent area of the transverse section, per unit width shall be given by:

$$A_x = t - \frac{\pi}{4P_x}(t_v)^2$$

- A_y, the equivalent area of the longitudinal section, per unit length shall be given by:

$$A_y = t\left[1 - 0.5\left(\frac{t_v}{t}\right)\left(1 + \frac{t_v}{t}\right) + 0.1\left(1.7 - \frac{t_v}{t}\right)\left(\frac{P_x - t_v}{t}\right)\right]$$

b) **Two-Dimensional Analysis of Wood Floor Systems**
In the absence of actual material properties, the following value of E_L shall be employed for the purpose of analysing wood floor systems:

$$E_L = 9600 \text{ MPa}$$

Values of E_T and G_{LT} shall be obtained from:

$$E_T = 0.05\,E_L$$
$$G_{LT} = 0.07\,E_L$$

where E_L is the modulus of elasticity in the principal direction shown in Figure A3—2(b), and G_{LT} is the shear modulus in the LT plane.
The six structural parameters required to idealize a wood floor system for two-dimensional analysis shall be calculated as shown in Table A3—2(d). The first footnote of Table A3—2(a) shall apply for the calculation of D_{xy} and D_{yx}.

Table **Properties of Idealized Orthotropic Plate or Grillage** **A3—2(a)**

	Properties to Define a Two-Dimensional Orthotropic Plate	Grillage Beam Properties To Define a Two-Dimensional Orthogonal Grillage
Longitudinal Direction	$D_x = Ei$	moment of inertia $I_x = i \times$ (longitudinal grillage beam spacing)
	$^*D_{xy} = Gi_0$	torsional inertia $J_x = i_0 \times$ (longitudinal grillage beam spacing)
	$D_1 = \nu$ (lesser of D_x & D_y)	No equivalent to D_1
Transverse Direction	$D_y = Ej$	moment of inertia $I_y = j \times$ (transverse grillage beam spacing)
	$^*D_{yx} = Gj_0$	torsional inertia $J_y = j_0 \times$ (transverse grillage beam spacing)
	$D_2 = D_1$	No equivalent to D_2
	$^{**}S_y = Gs$	transverse shear area = $s \times$ (transverse grillage beam spacing)

* There is a lack of consistency in the application of this value in various analyses. When the analysis utilizes the following expression for calculation of M_{xy}, the values of D_{xy} and D_{yx} shall be calculated as specified in the table.

$$M_{xy} = D_{xy}\frac{\partial^2 w}{\partial x\,\partial y}$$

However, when the following expression for calculating M_{xy} is used, the value of D_{xy} and D_{yx} shall be taken as half of that specified in the table.

$$M_{xy} = 2\,D_{xy}\frac{\partial^2 w}{\partial x\,\partial y}$$

For grillage beam properties, the expressions given in this table for torsional inertia are always correct as stated.

** Required only for voided slabs with rectangular voids.

Table **Calculation of Structural Parameters** **A3 — 2(b)**

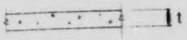
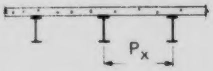
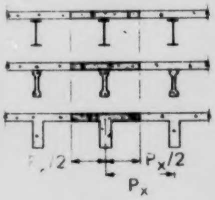
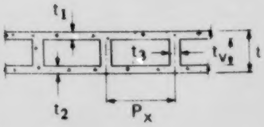
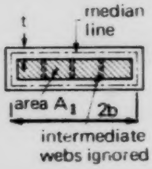
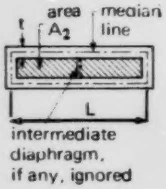
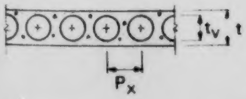
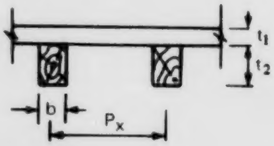
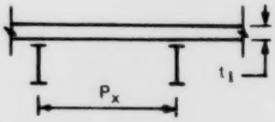
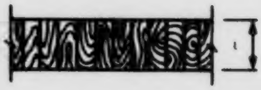
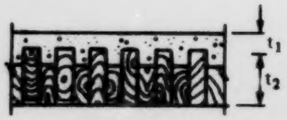
Bridge Type and Transverse Section	Structural Parameters in Slab Concrete Units i	j	i_o	j_o	s
Slab Bridge 	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}$	$\frac{t^3}{6}$	may be ignored
Non-composite slab-on-girder bridge 	$\frac{t^3}{12}+\frac{\text{moment of inertia of girder}}{P_x}$	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}$	$\frac{t^3}{6}$	may be ignored
Composite slab-on-girder bridge 	$\frac{\text{combined moment of inertia of hatched slab portion and girder}}{P_x}$	$\frac{t^3}{12(1-\nu^2)}$	$\frac{t^3}{6}+\frac{\text{torsional inertia of girder}}{P_x}$ **Note**: The torsional inertia of a steel girder may be ignored	$\frac{t^3}{6}$	may be ignored
Voided slab with rectangular voids  t_1 t_3 t_v t t_2 P_x	$\frac{t^3-t_v^3}{12}$	$\frac{t^3-t_v^3}{12}$	$\frac{4A_1^2}{2b\,\Sigma ds/t}$  t median line area A_1 2b intermediate webs ignored	$\frac{4A_2^2}{L\,\Sigma ds/t}$  t area A_2 median line L intermediate diaphragm, if any, ignored	$\frac{t_1^3+t_2^3}{P_x^2}\frac{E}{G}\times\frac{t_3^2P_x}{t_3^3P_x+(t_1^3+t_2^3)\left(\frac{t+t_v}{2}\right)}$
Voided slab with circular voids 	$\frac{t^3}{12}-\frac{\pi t_v^4}{64P_x}$	$F_1t^3/12$ F_1 from Figure A5—3(a)	$F_2t^3/6$ F_2 from Figure A5—3(a)	$F_2t^3/6$ F_2 from Figure A5—3(a)	may be ignored

Table **Torsional Constants for Rectangular Sections** **A3 — 2(c)**

a/b	1.0	1.2	1.5	2.0	2.5	3	4	5	10	∞
K	0.141	0.166	0.196	0.229	0.249	0.263	0.281	0.291	0.312	0.333

For $a \geqslant b$

Table — Calculation of Structural Parameters for Bridges Incorporating Wood Beams — A3 – 2(d)

Bridge Type and Transverse Section	Structural Parameters				
	D_x	D_y	D_{xy}	D_{yx}	$D_1 = D_2$
Transverse Laminated Decks on Longitudinal Wood Beams 	$E_L \frac{bt_2^3}{12P_x}$	$E_L \frac{t_1^3}{12}$	$G_{LT} \frac{Kt_2b^{3*}}{P_x}$	0.0	0.0
Transverse Laminated Decks on Longitudinal Steel Beams 	$\frac{E_s I}{P_x}$ where: I = moment of inertia of a girder, in steel units	$E_L \frac{t_1^3}{12}$	0.0	0.0	0.0
Glue Laminated and Transversely Prestressed Decks 	$E_L \frac{t^3}{12}$	$E_T \frac{t^3}{12}$	$G_{LT} \frac{t^3}{6}$	$G_{LT} \frac{t^3}{6}$	0.0
Composite Concrete Slab on Longitudinally Laminated Wood Deck 	$E_c I$ where: I = combined moment of inertia of a unit width of concrete and wood using a modular ratio of $\frac{E_c}{E_L}$	$E_c \frac{t_1^3}{12}$	$\frac{G_c t_1^3}{6}$	$G_c \frac{t_1^3}{6}$	νD_y

* K is obtained from Table A3 — 2(b)

4 Deck Joints and Bearings

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4 3 Notation [illegible]
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Reference Publications

Section 4 makes reference to the following publications:

CSA Standards	B95-1962	*Surface Texture (Roughness, Waviness and Lay)*
	CAN3-G40.21-M1981	*Structural Quality Steels*
ASTM Standards	A 167-80	*Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet and Strip*
	D 395-78	*Tests for Rubber Property — Compression Set*
	D 412-80	*Tests for Rubber Properties in Tension*
	D 429-73	*Tests for Rubber Property — Adhesion to Rigid Substrates*
	D 518-61	*Tests for Rubber Deterioration — Surface Cracking*
	D 573-81	*Test for Rubber — Deterioration in Air Oven*
	D 746-79	*Test for Brittleness Temperature of Plastic and Elastomers by Impact*
	D 1457-78	*Specification for TFE-Fluorocarbon Resin Molding and Extrusion Materials*
	D 2240-81	*Test for Rubber Property — Durometer Hardness*

Scope

4—1

This section provides requirements for the design and selection of bearings and deck joints, and for the articulation of the structure. The only deck joints considered are those at which rotation and/or translation can take place, construction joints being excluded.

Definitions

4—2

Articulation — a system of joints, bearings and restraints which limits or controls the movement of a structure.

Bearing — a structural device which transmits loads while permitting translation and/or rotation.

Deck Joint — a structural discontinuity between two elements, at least one of which is a deck element, which permits relative rotation and/or translation between the two.

Effective Rubber Thickness — the sum of the thicknesses of all the layers of elastomer which have a shape factor not greater than twelve.

Open Deck Joint — a deck joint which permits the free flow of water through the joint.

Sealed Deck Joint — a deck joint which prevents the passage of water through the joint.

Shape Factor — the area of the loaded face of an elastomeric layer divided by the area free to bulge.

Notation

4—3

H	the free height of a TFE element, mm.
L	the lesser plan dimension of a rectangular bearing, mm.
L_1	the greater plan dimension of a rectangular bearing, mm.
R	the radius of a circular bearing, mm.
S	the shape factor.
TFE	tetrafluoroethylene polymer.
T_e	the effective rubber thickness, mm.

Movements and Loads

4—4

Design Requirements

4—4.1

4—4.1.1 The number of deck joints and bearings in a structure shall be the practical minimum.

4—4.1.2 Deck joints and bearings shall be designed to resist loads and accommodate movements at the serviceability limit states Type II and at the ultimate limit states. The performance factor, ϕ, applied to the guaranteed minimum capacity of bearings shall be 1.0. Sealed joints shall remain watertight at the serviceability limit states Type II. Any damage to deck joints resulting from movements at the ultimate limit states shall be such that it can be repaired while the bridge remains in service.

Sources of Movements and Loads

4—4.2 The movements and loads shall be as specified in Section 2 and shall be calculated using the load combinations and load factors specified in Table 2—5.1(a).

The following factors shall be considered in proportioning and detailing bearings and deck joints.

a) properties of the materials in the structure, including coefficient of thermal expansion, modulus of elasticity, Poisson's ratio, creep and shrinkage;
b) effective temperature range of the structure;
c) sizes of the structural members;
d) method and sequence of construction;
e) tilt, settlement, and movement of supports;
f) construction tolerances;
g) static and dynamic response of the structure;
h) interaction of the forces to which the structure may be subjected, including dead and live loading, wind, earthquake and earth pressures;
i) structural restraints.

Deck Joints

4—5

General Requirements

4—5.1

4—5.1.1 Deck joints shall consist of components arranged so as to accommodate the translation and rotation of the structure at the joint. The joint shall provide for the passage of traffic across the joint and shall not significantly impair the riding characteristics of the roadway. Deck joints shall be detailed to prevent damage to the deck, bearings, piers and abutments from water, de-icing chemicals and roadway debris.

4 — 5.1.2 Deck joints shall be approved and shall be designed to operate with a minimum of maintenance for the design life of the structure. Consideration shall be given to the following requirements.

a) All materials in the joint shall be durable.
b) No surface of the joint exposed to traffic shall cause skidding and all parts shall be resistant to attrition and the impact of traffic, including snowplows.
c) The joint shall not amplify traffic generated noise and the components shall not translate nor rotate under the direct application of wheel loads.
d) The joint shall be accessible for inspection and maintenance and shall be replaceable except for anchorages permanently attached to the structure.
e) The joint shall be constructed so that it can be elevated, by modification or addition, to accommodate future overlays.

4 — 5.1.3 The edges of the roadway surfaces at the joint shall be protected by structural steel components which shall be designed for the loads specified in Section 2 and anchored in accordance with Clause 4 — 5.3.

4 — 5.1.4 Gaps at deck joints, with the exception of finger plate joints, measured perpendicular to the joint at the deck surface, at serviceability limit states Type II, shall not exceed:

a) 100 mm for deck joints with a single opening, or
b) 80 mm for each opening in deck joints with multiple openings.

Gaps in deck joints with multiple openings shall be equal.

Installation **4 — 5.2**

4 — 5.2.1 The plans shall include, in tabular form, the required installation openings throughout the probable installation temperature range. The installation temperature shall be as defined in Clause 4 — 5.2.2. The openings shall be calculated taking into account all anticipated translation and rotation. The effect of creep, shrinkage, elastic shortening, irreversible movements and the sequence of construction shall be taken into account.

4 — 5.2.2 In the absence of a more accurate procedure, the installation temperature shall be taken as the mean shade air temperature at the structure for the 48 hours prior to joint installation for concrete structures, and for the 24 hours prior to joint installation for structures where the main members are made of structural steel.

4 — 5.2.3 All components of the joint shall be recessed a minimum of 5 mm from the adjacent surface of the deck, curb, sidewalk, parapet or barrier wall which is exposed to the action of snowplows or other maintenance equipment. All cover plates shall be installed with the free end pointing in the direction of the traffic.

4 — 5.2.4 Where necessary, a rigid bedding course shall be provided to ensure complete contact between the bottom of the joint and the deck surface. The bedding material shall be plastic at the time of installation. The joint shall be placed before the bedding material sets and sufficient load shall be applied to squeeze out excess material and seat the joint at the elevations shown on the plans.

Anchorage **4 — 5.3**

4 — 5.3.1 Each side of the deck joint assembly shall be attached to the deck or abutment by anchoring the joint assembly so as to transfer all static and dynamic loads from the joint assembly to the structure. The following minimum requirements shall be satisfied. The performance factor, ϕ, shall be taken as 0.85.

a) Where the deck joint assembly is attached by reinforcing bars, studs or bolts cast into concrete, the total cross-sectional area of the steel anchors on each side of the joint shall be not less than 1600 mm^2/m; the spacing of the bars, studs or bolts shall not exceed 250 mm, and the factored resistance shall be not less than 600 kN/m in any direction.
b) Where the joint assembly is attached directly to the steel superstructure by welding or bolting, the weld or bolts shall have a factored resistance of not less than 600 kN/m in any direction. The spacing of bolts or the centre-to-centre distance of welds shall not exceed 250 mm.
c) Where the joint assembly is attached by a combination of anchorage in concrete and connection to steel members, the factored resistance of the anchorage shall be at least equal to (a) or (b) above.

4 — 5.3.2 Where bolts or welds are used to assemble components of a deck joint in direct contact with the wheels of vehicles, the factored horizontal resistance of the bolts or welds shall be not less than that resulting from the application of a factored force of 84 kN/m applied at the roadway surface in combination with forces which result from the movement of the joint to produce maximum force effects.

Open Deck Joints

4—5.4 Open deck joints shall permit the free flow of water through the joint. The deck joint, piers, and abutment shall be detailed to prevent the accumulation of water and debris which may restrict the movement of the structure. Positive protection against corrosion of embedded reinforcing steel shall be provided for all elements exposed to surface run-off containing de-icing chemicals.

Sealed Deck Joints

4—5.5

4—5.5.1 Sealed deck joints shall seal the surface of the deck, including curbs, sidewalks, medians, and, where necessary, parapet or barrier walls against the seepage of moisture and other contaminants through the joint. The deck joint shall prevent the accumulation of water and roadway debris which may restrict the operating capabilities of the joint.

4—5.5.2 All components of the structure shall be proportioned to resist the forces induced by the deck joints.

4—5.5.3 Each deck joint shall be prefabricated and assembled as a single unit wherever possible. Where field splices are necessary, the locations shall be shown on the plans.

4—5.5.4 Where the movement of the joint is accommodated through the change in geometry of an elastomer gland or a membrane, the gland or membrane shall not be in direct contact with the wheels of vehicles.

Bearings

4—6

General Requirements

4—6.1

4—6.1.1 Bearings shall consist of components arranged so as to transmit all factored loads and accommodate the factored translations and rotations of the structure.

4—6.1.2 Provision for translation shall be by means of rollers, rockers, shear deformation of an elastomer or through the provision of a sliding surface of a copper alloy or TFE, or other approved method. Provision for rotation shall be by means of cylindrical or spherical sliding surfaces, confined or unconfined elastomers, hinges, rollers or rockers or other approved method.

4—6.1.3 Bearings which do not meet the requirements of Clause 4—6 shall not be used unless approved.

4—6.1.4 The following maximum and minimum factored loads and movements corresponding to the critical combinations at the ultimate limit states and at the serviceability limit states Type II shall be shown on the plans:

a) axial dead load,
b) axial total load,
c) lateral loads,
d) rotations,
e) translations,
f) any other requirements which must be satisfied.

4—6.1.5 Bearings shall be constructed from materials which are durable and will perform the intended function for the design life of the structure.

4—6.1.6 The bearing shall be accessible for inspection and maintenance and shall be replaceable without damage to the structure and without removing anchorages permanently attached to the structure.

Installation

4—6.2

4—6.2.1 For bearings which have a sliding element the plans shall include, in tabular form, the required settings throughout the probable temperature range at the time of erection.

4—6.2.2 Bearings with a TFE sliding surface shall be plant assembled in such a way as to prevent separation of the sliding surfaces. The plans shall require that temporary connections not be removed until the bearing is in its final position, with all permanent connections made, and adjacent grout beds and concrete sections have been placed.

4—6.2.3 Where a bedding grout is used to seat the bearing, the grout shall be inert, free from shinkage and staining, and shall have a thickness of no more than 10 mm. Permanent shims less than the size of the bearing shall not be used.

Rollers, Rockers and Sliding Plates

4—6.3

4—6.3.1 All steel components shall be proportioned in accordance with the requirements of Section 10. Bronze and copper sliding plates shall be in accordance with Clause

4—6.3.6. TFE sliding surfaces shall be in accordance with Clause 4—6.4.

4—6.3.2 Rollers shall have a minimum diameter of 200 mm. Roller nests shall consist of no more than two rollers and shall be detailed to ensure equal loading on each roller.

4—6.3.3 Rollers, rockers and sliding plates shall be secured against lateral movement by the provision of dowels or guide bars. Rollers and rockers shall be secured against skewing and creeping by the provision of pintles or a rack and pinion mechanism. Base plates shall be secured against movement.

4—6.3.4 The upper surface of a rocker shall have a pin or cylindrical bearing, and the lower surface shall be cylindrical. The centre of rotation of the upper surface shall not be higher than the centre of rotation of the lower bearing surface. Where the two centres of rotation do not coincide, the resulting longitudinal force shall be included in the analysis of the structure and its components. The rocker and the contact surface shall be designed to ensure uniformly distributed loads. Cantilevers which are not stiffened to ensure uniformly distributed loads shall be neglected in calculating the load-carrying capacity of the rocker.

4—6.3.5 The coefficient of friction shall be determined on the basis of test data provided by the manufacturer with due consideration given to the anticipated change in the coefficient of friction during the life of the structure.

4—6.3.6 Copper alloy sliding plates shall be not less than 10 mm thick unless the plate is bonded to a structural backing plate. The sliding plates shall be securely held in position, shall be protected against the accumulation of foreign material, and shall be chamfered at the ends.

TFE Sliding Surfaces 4—6.4

4—6.4.1 **General**

TFE sliding surfaces shall be designed to translate by sliding of a stainless steel mating surface against a TFE element.

4—6.4.2 **TFE**

The TFE resin shall be virgin material having a relative density of 2.16 ± 0.03, and shall be in accordance with the requirements of ASTM Standard D1457. The resin may be used as filled or unfilled sheets. The deviation from flatness of the TFE shall not exceed 0.2 mm when the diameter or diagonal is less than 800 mm nor 0.00025 of the diameter or the diagonal when this dimension is greater than 800 mm.

4—6.4.3 **Contact Surface**

The contact surface shall be stainless steel having a corrosion resistance not inferior to that required by ASTM Standard A167 Type 304. The contact surface shall be plane, cylindrical, or spherical. The roughness of the contact surface, measured in accordance with CSA Standard B95, shall be not greater than 0.25 μm arithmetic average for plane surfaces and 0.5 μm arithmetic average for curved surfaces. The deviation from flatness for plane surfaces and from the theoretical surface for curved surfaces shall not exceed $0.0003L_1H$ mm for a rectangular TFE element nor $0.0006RH$ mm for a circular TFE element. The thickness of the stainless steel sheet shall be not less than that given in Table 4—6.4.3 for the dimensional difference between the stainless steel and the TFE in the direction of movement.

Table **Thickness of Stainless Steel Sheet** **4—6.4.3**

Dimensional Difference Between TFE and Stainless Steel, mm	Minimum Thickness of Stainless Steel, mm
≤300	1.5
> 300 and ≤ 500	2.0
> 500 and ≤ 1500	3.0

4—6.4.4 **Lubrication**

All sliding surfaces, except those which serve as guides and are subject to a constant pressure of less than 5 MPa, shall be permanently lubricated. Spherical reservoirs for the lubricant shall be provided in the surface of the TFE. The diameter of the reservoirs shall not exceed 8 mm, measured at the surface of the TFE, and the depth shall be not less than 2 mm nor more than half the thickness of the TFE. The reservoirs shall be evenly distributed across the surface of the TFE and shall occupy not less than 20% of its surface. The lubricant shall be silicone grease, effective to -35°C.

4 — 6.4.5

Design Requirements

The TFE sliding element shall be fully bonded in the manufacturer's plant, and recessed in a rigid backing material. The thickness and free height of the TFE element shall be not less than those given in Table 4 — 6.4.5.

Table **Thickness and Free Height of TFE Elements** **4 — 6.4.5**

Maximum Plan Dimension, mm	≤ 600	> 600 and ≤ 1200	>1200
Thickness, mm	4.5	5.0	6.0
Free Height, mm	2.0	2.5	3.0

4 — 6.4.6

Contact Pressures

The average contact pressure for TFE sliding elements shall not exceed the values given in Table 4 — 6.4.6. The maximum contact pressures shall not exceed 1.2 times those given in Table 4 — 6.4.6. For calculation of pressures, the contact surface shall be taken as the gross area of the TFE without deduction for the area occupied by the lubrication cavities.

Table **Average Contact Pressures for TFE Sliding Elements** **4 — 6.4.6**

Limit State	Permanent Load (MPa)	All Loading Combinations (MPa)
Serviceability Type II	30	45
Ultimate	45	70

4 — 6.4.7

Coefficient of Friction

The coefficient of friction used in the design of an element shall be that value between 0.005 and the value prescribed in Table 4 — 6.4.7, which produces the critical response.

The coefficient of friction may be interpolated linearly from the values given in Table 4 — 6.4.7 for contact pressures within the range given.

Table **Coefficient of Friction for Lubricated TFE Sliding Surfaces** **4 — 6.4.7**

Contact Pressure, MPa	Coefficient of Friction	
	Unfilled	Filled
10	0.06	0.12
15	0.05	0.10
20	0.04	0.08
25	0.03	0.07
30 and above	0.03	0.06

Unconfined Elastomeric Bearings 4 — 6.5

4 — 6.5.1

General

Unconfined elastomeric bearings may be plain bearings consisting entirely of an elastomer, or steel reinforced bearings consisting of alternate layers of elastomer and steel. Materials for bearings shall be in accordance with Appendix A4 — 1 and bearings shall be manufactured in accordance with the requirements of Appendix A4 — 2.

The shear resistance of the bearing shall not exceed 240 kPa for polyisoprene bearings, nor 350 kPa for polychloroprene bearings when tested in accordance with Appendix A4 — 3.

The compressive deformation of laminated bearings shall not exceed $0.04T_e$ when tested in accordance with Appendix A4 — 4.

4 — 6.5.2 **Geometrics**

4 — 6.5.2.1 The bearing areas of the structure shall be detailed to ensure complete contact with the bearing under all loading combinations.

4 — 6.5.2.2 Bearings shall have the following proportions:

a) plain bearings:

$$L \geqslant 5\,T_e$$
$$R \geqslant 3\,T_e$$

where: $10\text{ mm} \leqslant T_e \leqslant 30\text{ mm}$

b) laminated bearings:

$$L \geq 3\,T_e$$
$$R \geq 2\,T_e$$

4 — 6.5.3 **Performance**

4 — 6.5.3.1 The shear deformation of the bearing shall not exceed $0.5T_e$ at serviceability limit states Type II.

4 — 6.5.3.2 Positive attachment to prevent displacement of the bearing shall be provided if any of the following conditions exist:

a) the bearing may be displaced during construction;

b) the shear force generated by the bearing exceeds the frictional resistance between the structure and the loaded faces of the bearing;

c) the minimum average pressure on the bearing is less than 1.5 MPa at serviceability limit states Type II, or less than 2.0 MPa at the ultimate limit states.

4 — 6.5.3.3 The average pressure on a laminated bearing shall not exceed the following:

a) Serviceability Limit States Type II:
   4.5 MPa under permanent load,
   7 MPa under all loading combinations;

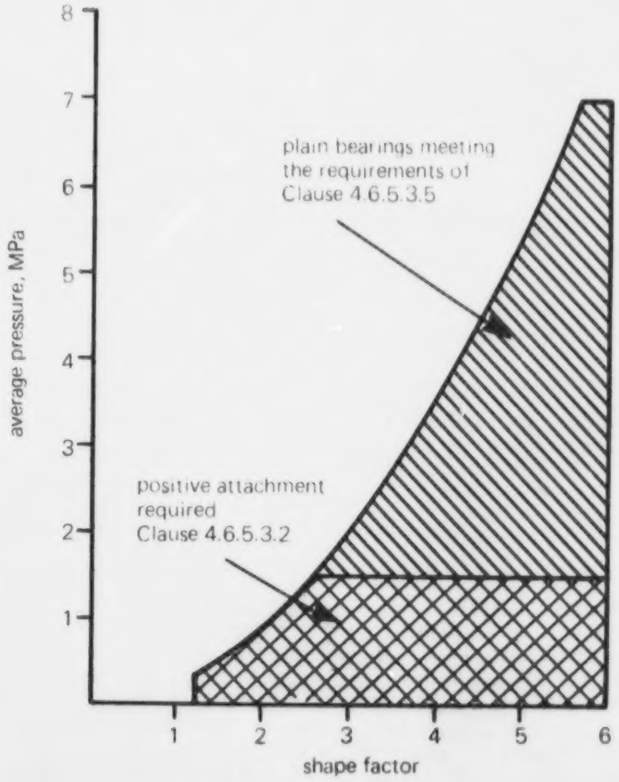
b) Ultimate Limit States:
   7 MPa under permanent load,
   10 MPa under all loading combinations.

4 — 6.5.3.4 The rotation of a laminated bearing at serviceability limit states Type II shall be such that under minimum load it does not result in an uplift at the edge of the bearing, and under maximum load the compressive deformation does not exceed $0.06T_e$.

4 — 6.5.3.5 The maximum average pressure on a plain bearing at the serviceability limit states Type II, and where there is no rotation, shall not exceed the following:

a) that specified in Figure 4 — 6.5.3.5 corresponding to the shape factor of the bearing, and

Figure **Maximum Average Pressure on a Plain Bearing at SLS II Without Rotation** **4 — 6.5.3.5**



b) 4.5 MPa under permanent load

Where rotation occurs, the bearing shall be designed so that at ultimate limit states and at the serviceability limit states Type II there is no uplift and, at the serviceability limit states Type II, the edge deformation does not exceed $0.07T_e$ under the maximum load.

Confined Elastomer Bearings **4 — 6.6**

4 — 6.6.1 The material of the elastomeric component shall be in accordance with Appendix A4 — 1.

4 — 6.6.2 At serviceability limit states Type II and maximum rotation, the eccentricity of the axial load shall not exceed 3% of the diameter of the elastomeric disc.

Structure Articulation **4 — 7**

Where bearings are provided, they shall be part of a complete and effective system of articulation. The structure shall be anchored where the analysis shows that restraint is required so that the structure can translate and rotate without damage or permanent displacement.

Appendix **Material Requirements for Elastomeric Bearings** **A4 — 1**

The requirements for the elastomeric components of confined and unconfined elastomeric bearings and for the steel plates in unconfined laminated elastomeric bearings shall include the following:

a) No reclaimed materials shall be incorporated in the finished bearing.

b) Virgin natural polyisoprene or virgin polychloroprene shall be the only raw polymers permitted.

c) The physical properties of vulcanized elastomer shall be determined using test specimens taken from sample bearings.

d) The physical properties of any polyisoprene and polychloroprene used shall meet the requirements of Table A4 — 1, Columns 1 and 2, respectively. A 10% variation in the tensile strength and ultimate elongation values shall be allowed when specimens are taken from field samples.

e) Internal steel plates shall be not less than 3 mm thick and not more than 5 mm thick. All steel shall conform to CAN3-G40.21 Grade 230G.

Appendix **Manufacturing Requirements for Unconfined Elastomeric Bearings** **A4 — 2**

The requirements for the manufacture of unconfined elastomeric bearings are as follows:

a) Moulds

All moulds shall have a smooth surface finish.

b) Dimensional Tolerances

Dimensional tolerances shall be as follows:

i) When the average total elastomer thickness ≤ 30 mm, the overall vertical dimensional tolerance shall be −0, +3 mm.

ii) When the average total elastomer thickness > 30 mm, the overall vertical dimensional tolerance shall be −0, +6 mm.

iii) Tolerance on overall horizontal dimension shall be −0, +6 mm.

iv) Tolerance on thickness of individual layers of elastomer shall be ±20%.

v) Permitted variation from plane parallel to theoretical surface shall be 1 in 200 for tops, 1 in 100 for sides, and $0.25\,T_e$ for steel laminates when tested in accordance with Appendix A4 — 5.

vi) Tolerance on edge cover of embedded steel shall be ±2 mm.

vii) Tolerance for internal holes or slots shall be ±3 mm.

viii) Tolerance for the relative position of holes, slots or inserts to each other shall be ±2 mm.

c) Plain Bearings

Plain bearing pads shall be moulded individually, cut from previously moulded strips or slabs of the required thickness, or extruded and cut to length.

d) Steel-Laminated Bearings

Steel-laminated bearings shall be moulded as a single unit under pressure and heat, and the steel laminates shall conform to the following:

i) All steel plates shall be of uniform thickness.

ii) Internal steel plates shall be free from sharp edges.

iii) Steel laminates shall be completely bonded on all surfaces to the elastomeric material during moulding. The edge cover shall be 5 mm except that no elastomer cover is required over internal details (i.e., vertical holes or slots covered by bearing seats or flanges and which will not be exposed to moisture after erection of the bridge).

e) Elastomer Laminates

All internal elastomer laminates shall be of uniform thickness.

Appendix **Method of Test for the Shear Resistance of Unconfined Elastomeric Bearings** **A4—3**

For establishing the resistance to shear of plain and laminated unconfined elastomeric bearings, the method of test shall be as follows.

a) The bearings shall be conditioned at –25C ± 2°C for 95 h ± 1 h under a compressive stress of 3.5 MPa and a shear strain of 15% of the effective rubber thickness.

b) The bearing shall be deformed to a shear strain of 25% of the effective rubber thickness while maintaining the compressive stress of 3.5 MPa.

c) The force required to maintain the shear strain of the effective rubber thickness 15 minutes after the strain is achieved, shall be recorded.

Appendix **Method of Test for the Compressive Deformation of Laminated Bearings** **A4—4**

For the compressive deformation of steel-laminated, unconfined elastomeric bearings, the method of test shall be as follows.

a) The testing machine used shall have platens at least 20 mm greater in both plan dimensions than the bearing under test.

b) At least two dial gauges shall be positioned at the centre of opposite sides of the bearing to measure strain.

Table **Material Requirements for Elastomeric Bearings** **A4.1**

Property	Test	Column 1,	Column 2,
		Polyisoprene Requirements	Polychloroprene Requirements
Hardness °, Shore A	ASTM D2240	55 ± 5	55 ± 5
Tensile strength, MPa	ASTM D412	min. 17	min. 17
Ultimate elongation, %	ASTM D412	min. 400	min. 400
Heat resistance	ASTM D573	70 h at 70°C	70 h at 100°C
Change in hardness °, Shore A		max. +10	max. +15
Change in tensile strength, %		max. –25	max. –15
Change in ultimate elongation, %		max. –25	max. –40
Compression set, %	ASTM D395 Method B	22 h at 70°C max. 25	22 h at 100°C max. 35
Ozone	ASTM D518 mounting Procedure A 20% strain, 40 ± 2°C	25 pphm, 48 h no cracks	100 pphm 100 h no cracks
Bond during vulcanization, kPa	ASTM D429 Method B	min. 350	min. 350
Brittleness at –40°C	ASTM D746 Procedure B	no failure	no failure
Low temperature crystallization increase in hardness °, Shore A	ASTM D2240	168 h at –25°C max. +15	168 h at –10°C max. +15

c) The readings on the dial gauges shall be recorded.
d) The bearing shall be loaded at the rate of 1.5 MPa/min to a load of 7.5 MPa multiplied by the gross plan area; the dial gauge reading shall be recorded.
e) The bearing shall be unloaded at the same rate until the pressure on the bearing is 1.5 MPa, and the dial gauge readings recorded.
f) The pressure on the bearing shall be maintained at 1.5 MPa for 15 minutes, and the dial gauge readings shall be recorded before and after the 15-minute test interval.
g) The bearing shall be reloaded to 7.5 MPa and steps (e) and (f) shall be repeated.
h) The bearing shall be reloaded to 8 MPa, with the dial gauge readings being recorded after each 500 kPa increment.
i) A graph of presssure versus average deformation from the data recorded in (h) shall be constructed. The change in deflection between 1.5 and 7 MPa on the third loading shall be reported.

Appendix **Method of Test for the Tolerance of Steel Laminates** **A4 — 5**

For the tolerance of the steel laminates in steel-laminated unconfined elastomeric bearings, the method of test shall be as follows.

a) The distance from the base of the bearing to the bottom of every plate shall be measured. Measurements shall be taken at each corner of rectangular bearings and at the extremities of two perpendicular diameters of circular bearings.
b) The difference between the highest and lowest measurements for every plate shall be recorded.
c) The cumulative total of the differences recorded, expressed as a fraction of the effective rubber thickness of the bearing, shall be reported.

5 Accessories

Clause	Contents	
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5.2	**Definitions**	[illegible]
5.3	**Notation**	[illegible]
5.4	**General Features of Supports**	[illegible]
5.4.1	General	[illegible]
5.4.2	Location of Supports	[illegible]
5.4.3	Vertical Clearance	[illegible]
5.5	**Limit States for Supports**	[illegible]
5.5.1	Ultimate Limit States Design	[illegible]
5.5.2	Serviceability Limit States Design	[illegible]
5.6	**Breakaway Supports**	[illegible]
5.6.1	Usage and Function	[illegible]
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Reference Publications

Section 5 makes reference to the following publications:

CSA Standards

CAN3-G40.21-M81	*Structural Quality Steel*
G164-M1981	*Hot-Dip Galvanizing of Irregularly-Shaped Articles*
G189-1966(R1980)	*Sprayed Metal Coatings for Atmospheric Corrosion Protection*
S157-M	*Strength Design in Aluminum* (To be Published)

Scope

5—1

This section specifies requirements for the design of supports for highway signs, barrier walls, luminaires, traffic signals and also bridge railings and other appurtenances.

Definitions

5—2

Auxiliary Lane — the portion of a roadway adjoining the through lanes for speed change lanes, truck climbing lanes or other purposes supplementary to through traffic movement.

Barrier Wall — a fully closed fence providing an audio or visual barrier; or a wall, with or without railings, intended to confine vehicles and pedestrians.

Base Mounted Support — a support having a base which is fastened to a concrete foundation or other structure by means of anchor bolts or similar devices.

Breakaway Support — a support designed to fail when struck by a vehicle, such that damage to the vehicle and injury to the occupants is minimized.

Bracket Arm — a member or truss connecting a luminaire or traffic signal to a vertical pole or other support.

Cantilevered Sign — a sign mounted on a support located beyond the edge of the shoulder or driving lane and extending over the shoulder or driving lanes.

Combination Railing — a railing to contain both vehicular and pedestrian traffic.

Direct Embedment Support — a support, the lower end of which is placed in an excavated hole which is then backfilled with soil, concrete or other material.

Gore — the paved or unpaved area between the shoulders of two converging roadways or of a roadway and a converging ramp.

Guiderail — a barrier located along the edge of a roadway shoulder for the purpose of guiding errant vehicles.

Lane — a portion of the roadway for the movement of vehicles.

Luminaire — a complete lighting fixture, including the light source, reflector, refractor, housing and ballast where present, but excluding support members.

Overhead Sign — a sign mounted on a support spanning over a roadway and which is supported on both sides of the roadway.

Rail — a horizontal railing component spanning between railing supports.

Railing — a barrier consisting of rails and supports, or a barrier wall intended to confine vehicles and pedestrians.

Roadside Sign — a sign adjacent to a roadway with no part of the support or the sign extending over the roadway.

Roadway — the portion of a highway, including shoulders, for vehicular use.

Service Walk — a platform designed to provide access to signs and to sign illumination systems for maintenance purposes.

Sign — a panel displaying a message.

Support — a metal, concrete or timber structure including supporting brackets, service walks and mechancial devices where present, which is designed to support a luminaire, sign, or traffic signal.

Tenon — a sleeve which fits into the mounting aperture of a luminaire or traffic signal and which is permanently attached to a support.

Traffic Signal — a complete signal device consisting of traffic lights and housing and, where present, a backboard.

Travelled Way — the portion of a roadway for the movement of vehicles excluding the auxilliary lanes and shoulders.

Notation

5—3

b	width or diameter of footing, m.
D	depth of embedment, m.
H	height of load P above ground, m.
k	number of posts supporting a single rail element.
L	post spacing in a bridge railing measured centre-to-centre of posts, mm; or span of an overhead sign support, mm; or length of cantilever of a cantilevered sign support or traffic signal support, mm.
L_D	deck drain length in plan, mm.
n	number of traffic rails in a combination railing.
P	traffic load on railings, kN; or resultant of overturning forces on a support, kN.
S	average lateral soil pressure over top two-thirds of footing embedment, kPa.
S_1	computed lateral soil pressure at one-third depth of footing embedment, kPa.
S_2	computed lateral soil pressure at two-thirds depth of footing embedment, kPa.
W_D	deck drain width in plan, mm.
W_p	pedestrian load on railings, kN/m.

General Features of Supports 5—4

General 5—4.1

5—4.1.1 Supports shall be as shown schematically in Figures 5—4.1(a), 5—4.1(b), 5—4.1(c), and 5—4.1(d). Bridges may also serve as supports. Other types of supports shall not be used unless approved.

5—4.1.2 The visual impact of signing and lighting installations shall be considered, with due regard for the surrounding environment. Supports shall be designed to have clean simple lines and to minimize hazards.

5—4.1.3 The number of supports along the highway shall be kept to a minimum by such means as the use, whenever practicable, of poles which serve multiple purposes.

Location of Supports 5—4.2

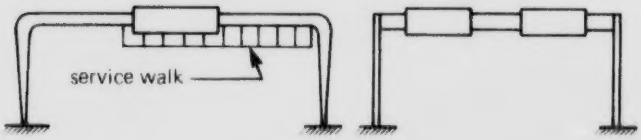
5—4.2.1 The location of supports shall be based upon safety considerations directed at minimizing the risk of the supports being struck by vehicles. Supports shall be placed so as to avoid distracting the motorist's attention and to avoid obstructing his view of the highway and of other signs or signals. Their locations shall conform to the requirements of Figure 5—4.2.1.

5—4.2.2 On roadways with a posted speed limit of 70 km/h or more, supports with a horizontal clearance of less than 10 m to the edge of the travelled way shall be protected by guiderail, barrier walls, retaining walls or an approved alternative form of vehicle restraint or, where such protection is not practicable, shall be breakaway supports.

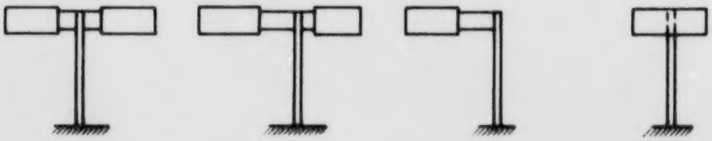
5—4.2.3 On roadways with a posted speed limit of less than 70 km/h supports shall be protected by guiderail or an approved alternative form of vehicle restraint. Where such protection is not practicable, a support shall have a horizontal clearance of not less than 600 mm from:

- a curb with a height of not less than 150 mm for supports for overhead signs and cantilevered signs; or,
- a curb or the edge of shoulder for all other supports.

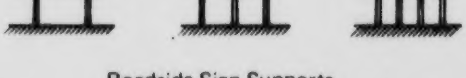
Figure **Sign Supports** **5—4.1(a)**



Overhead Sign Supports

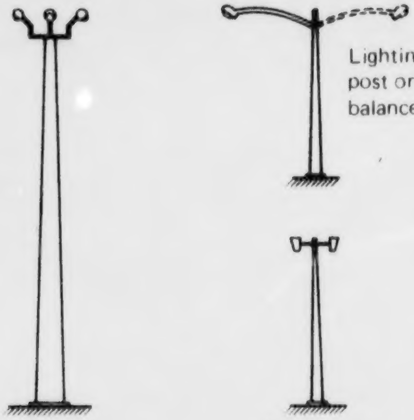


Cantilevered Sign Supports



Roadside Sign Supports

Figure **Luminaire Supports** **5 — 4.1(b)**

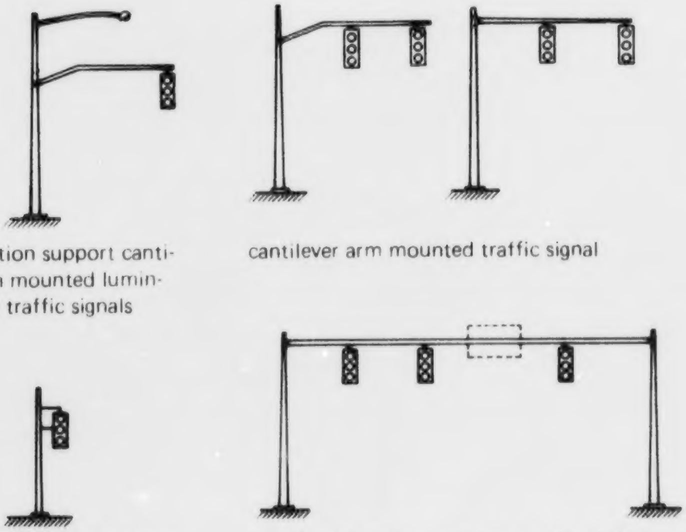


Lighting standard post or pole balanced or unbalanced

High mast lighting support

Floodlighting support

Figure **Traffic Signal Supports** **5 — 4.1(c)**



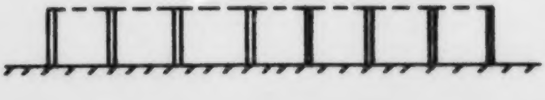
combination support cantilever arm mounted luminaires and traffic signals

cantilever arm mounted traffic signal

pole mounted traffic signal

traffic signals mounted on overhead sign support

Figure **Barrier Wall Supports** **5 — 4.1(d)**



5 — 4.2.4 Guiderail or other protection shall not be installed for the sole purpose of protecting supports for roadside signs, traffic signals or luminaires.

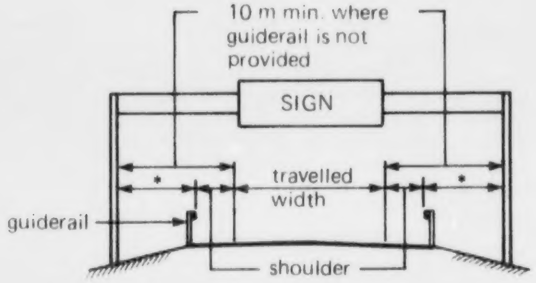
5 — 4.2.5 Breakaway supports shall not be used for overhead signs or for cantilevered signs.

5 — 4.2.6 Supports mounted on a bridge or retaining wall having a combination railing shall be placed outside the railing.

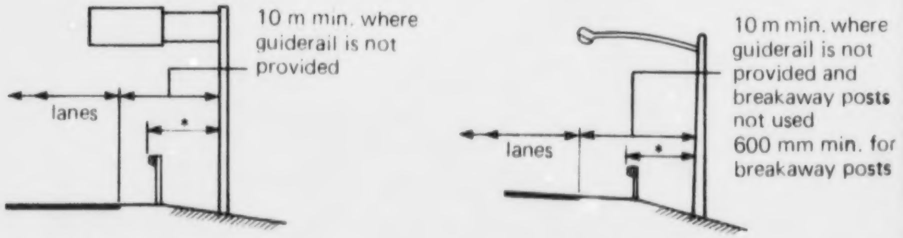
Supports for overhead signs spanning the deck of a bridge shall be mounted only on bridges with barrier wall railings.

5 — 4.2.7 Sign or luminaire supports shall not be located in a gore unless some other major obstruction in the gore is unavoidable.

Figure **Locations of Supports** **5 — 4.2.1**

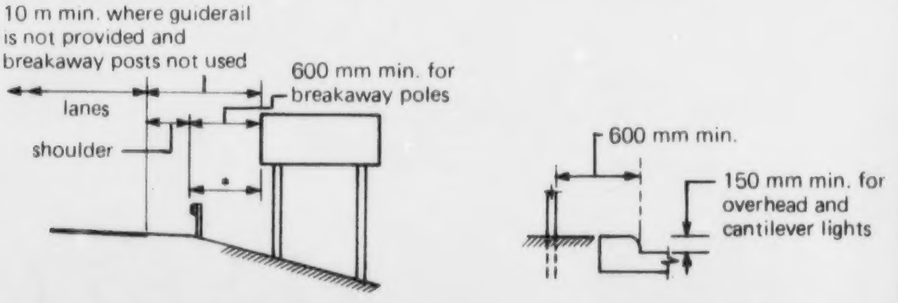

10 m min. where guiderail is not provided
SIGN
*
travelled width
*
guiderail
shoulder

Overhead sign support


10 m min. where guiderail is not provided
lanes
*
10 m min. where guiderail is not provided and breakaway posts not used
600 mm min. for breakaway posts
lanes
*

Cantilever sign support

Luminaire support


10 m min. where guiderail is not provided and breakaway posts not used
600 mm min. for breakaway poles
lanes
shoulder
*
600 mm min.
150 mm min. for overhead and cantilever lights

Roadside sign support

Support without guiderail, speed limit less than 70 km/h

* 300 mm plus design deflection of guiderail

Vertical Clearance

5—4.3

5—4.3.1 **Vertical Clearance for Sign Supports**

5—4.3.1.1 For signs other than those mounted in the side face of a bridge and those specified in Clause 5—4.3.1.2, the vertical clearance to the lowest projection of the support or sign, over the roadway, shall be not less than 5.30 m and shall also be at least 300 mm greater than the vertical clearance prescribed for bridges crossing over the highway.

For signs mounted on the side face of a bridge, no portion of the support or sign shall project below the soffit of the bridge unless approved.

5—4.3.1.2 Notwithstanding the requirement of Clause 5—4.3.1.1, a minimum clearance of 4.80 m may be used for small lane designation signs of light construction on overhead sign supports provided that a mechanism is included which will prevent damage to the overhead sign or to the sign support when a lane designation sign is struck by a vehicle.

5—4.3.2 **Vertical Clearance for Luminaire Supports and Traffic Signal Supports**

5—4.3.2.1 The vertical clearance from the roadway to a luminaire or the portion of the luminaire support located over the roadway shall be not less than 7.6 m.

5—4.3.2.2 The vertical clearance from the roadway to a traffic signal or the portions of the traffic signal support located over the roadway shall be not less than 4.50 m.

Limit States for Supports

5—5

Ultimate Limit State Design

5—5.1

5—5.1.1 Aluminum members and aluminum connections in supports shall be designed to conform to the requirements of the CSA Standard S157-M, *Strength Design in Aluminum*, and the factored resistances of the members and connections shall be computed as specified in that Standard, except that load factors shall be in accordance with Section 2 of this Code.

Members in other materials shall be designed in accordance with the appropriate sections of this Code.

5—5.1.2 **Castings**

Structural castings shall be subjected to appropriate acceptance tests.

A performance factor, ϕ, of 0.7 shall be used to determine the test load unless the validity of a higher factor is demonstrated.

The test load shall not be less than the load imposed on the casting by the application of the design load divided by ϕ.

5—5.1.3 **Anchor Bolts**

When anchor bolts are cast into concrete the factored resistance of the anchor bolts shall be not less than 1.2 times the factored resistance of the component being anchored. When threaded inserts for anchor bolts are used the factored resistance of the inserts shall be not less than 1.2 times the factored resistance of the anchor bolts.

Serviceability Limit States Design

5—5.2

5—5.2.1 **Serviceability Limit States Type I**

Structural castings shall be designed to avoid unusual stress raisers. A casting which satisfies the test requirements of Clause 5—5.1.2 is acceptable from the standpoint of fatigue considerations.

5—5.2.2 **Serviceability Limit State Type II**

At the serviceability limit state Type II, the horizontal deflection of traffic signal and luminaire supports not exceeding 16 m in height shall not exceed 0.05 times the length of the support, unless approved. The length of the support shall be taken as the total distance to the extremity of the bracket arm from ground level, or from the top of the base pedestal if present, and measured along the line of the vertical support member and the bracket arm. In computing horizontal deflections, the rotation of the foundation may be neglected provided that the requirements of Clause 5—7.2.5 are met.

Deflection limitations for supports exceeding 16 m in height shall be as approved.

Breakaway Supports 5—6

Usage and Function 5—6.1

Only those types of breakaway supports which pass approved tests or which have performed safely in actual service shall be used.

Vertical Clearance to Breakaway Roadside Sign Supports 5—6.2

5—6.2.1 The lowest edge of the sign shall be not less than 1.50 m above the elevation of the edge of pavement.

No horizontal support member shall be located below the lowest edge of the sign.

5—6.2.2 No portion of the sign shall be less than 1.35 m above the ground level immediately below the sign.

Placement of Footings for Breakaway Roadside Sign Supports 5—6.3

5—6.3.1 The top of concrete in footings shall be not more than 75 mm above ground level.

5—6.3.2 Where a footing is located on a slope, the centre of the top surface of the footing shall be placed at ground level, and the area around the footings shall be so graded that no part of the top surface of the footing is below ground nor is more than 75 mm above ground.

5—6.3.3. The top surface of concrete footings shall be domed in order to prevent water from ponding on top of the footing.

Details for Breakaway Roadside Sign Supports 5—6.4

Breakaway roadside sign supports shall be provided with the following:

a) a frangible or slip base which will permit the support column to disengage from the foundation upon impact by a vehicle, the plane of fracture or slip being located not more than 150 mm above ground level;

b) a zone or splice with limited moment capacity in each supporting column located at or near the lower edge of the sign, which will act as a hinge but not separate when the support is struck by a vehicle;

c) except for the uppermost one, all connections between each support column and the sign or horizontal support members shall be designed as frangible or deformable connections permitting each column to separate from its lower attachments if the hinge referred to in (b) above does not develop. The uppermost connection of the column to the sign or horizontal support member shall be designed to ensure that the column will remain attached after being struck by a vehicle;

d) a wood support that embodies an approved method of inducing a fracture plane is an acceptable alternative to requirement (a).

Spacing of Breakaway Sign Support Posts 5—6.5

Where a sign is supported on two columns, these shall be at least 2.40 m apart, centre-to-centre.

Where a sign is supported on three or more columns these shall be at least 1.50 m apart, centre-to-centre.

Details for Breakaway Luminaire Supports 5—6.6

5—6.6.1 Base mounted breakaway luminaire supports shall be provided with a frangible or slip base which will permit the support column to disengage from the foundation when struck by a vehicle, the plane of fracture or slip being located not more than 150 mm above ground level.

5—6.6.2 Direct embedment breakaway luminaire supports shall be provided with a frangible or slip insert or section which will permit the support column to disengage from the embedded portion when struck by a vehicle, the plane of fracture or slip being located not more than 150 mm above ground level.

Placement of Footings for Breakaway Luminaire Supports 5—6.7

Footings for breakaway luminaire supports shall comply with the requirements of Clauses 5—6.3.1, 5—6.3.2 and 5—6.3.3.

Foundations for Supports

5—7

Soil Data

5—7.1

5—7.1.1 For the following applications the soil properties used in the design shall be obtained from soils investigations carried out for each installation:

a) luminaire supports greater than 16 m in height
b) overhead sign supports carrying sign areas greater than 40 m² or changeable message signs with areas greater than 20 m²
c) supports with piled foundations
d) supports with drilled shaft foundations greater than 4 m in depth

5—7.1.2 In the design of supports other than those specified in Clause 5—7.1.1., applicable soils information available from investigations at neighbouring sites, soils borings for highway design or other appropriate sources may be used.

Foundation Stability and Design

5—7.2

5—7.2.1 Spread footings and piled footings for supports shall conform to the requirements of Section 6.

5—7.2.2 Where a widened top or cap on a drilled shaft is used to accommodate the anchor bolts of a support or to provide additional bearing area against the upper layer of soil, the cap shall be designed so that it can transmit the factored loads from the support base to the shaft.

The bottom of the cap shall be located below frost penetration depth unless provision is made to prevent frost pressures from being applied to the bottom of the cap.

5—7.2.3 Posts directly embedded in earth shall be designed as cantilevers fixed at a point one-third of the embedment depth below the final ground line.

5—7.2.4 Where concrete encasement around the embedded portion of a post does not extend up to ground level, the encased portion only shall be considered as the depth of embedment and the post shall be designed as a cantilever fixed at the top of the concrete encasement.

5—7.2.5 For direct embedment supports and for base mounted supports on drilled caisson footings with diameters not greater than one metre and lengths not greater than 3.0 m, and at the serviceability limit state Type II, the computed lateral soil pressures at one-third and two-thirds the depth of embedment below grade as computed by the following equations, respectively, shall not exceed permissible lateral bearing pressures at those elevations established by a qualified geotechnical engineer.

$$S_1 = 1.5S \qquad 5.7\,(1)$$

$$S_2 = \left[\frac{2H + 0.68D}{H + 0.9D}\right] S \qquad 5.7\,(2)$$

where $$D = \frac{1.18P}{bS} + \sqrt{\left(\frac{1.18P}{bS}\right)^2 + \frac{2.63\,PH}{bS}} \qquad 5.7\,(3)$$

Detailing of Supports

5—8

Corrosion Prevention

5—8.1

5—8.1.1 All steel components shall be fabricated from stainless or atmospheric corrosion resistant steel or shall be hot-dip galvanized or metallized.

The alloy grades of stainless steel used shall be such that corrosion in the roadside environment will not occur.

5—8.1.2 In the following applications all steel components shall be stainless steel or hot-dip galvanized structural steel:

a) overhead sign supports
b) cantilever sign supports
c) those members of breakaway sign supports which are directly involved in the breakaway function
d) any member which by rusting would produce unsightly stains on concrete faces visible to motorists or pedestrians.

5—8.1.3 Hot-dip galvanizing shall be in accordance with CSA Standard G164-M1981, *Hot-Dip Galvanizing of Irregularly-Shaped Articles*.

5—8.1.4 Metallizing shall be in accordance with CSA Standard G189 1966 (R1980), *Sprayed Metal Coatings for Atmospheric Corrosion Protection*.

5—8.1.5 Metal or concrete direct embedment supports shall be coated with a coal tar epoxy paint over the complete depth of embedment.

5—8.1.6 Aluminum surfaces which are to be placed in contact with concrete, wood or masonry shall be given a coat of alkali-resistant bituminous paint, at least 0.50 mm thick, before installation.

5—8.1.7 Aluminum shall not be placed in contact with steel other than stainless steel or hot-dip galvanized steel.

5—8.1.8 Fasteners for structural connections shall be stainless steel or hot-dip galvanized steel.

Materials and Minimum Thicknesses

5—8.2

5—8.2.1 **Steel**

5—8.2.1.1 Steel for supports, and anchor bolts, shall meet the requirements of Section 10.

5—8.2.1.2 The minimum thickness of material for the chords of steel truss supports shall be 4.5 mm, and for diagonal members and bracing the minimum thickness shall be 3 mm.

5—8.2.1.3 Pole type supports shall be of closed cross-section and the following minimum steel thicknesses shall apply:

a) 1.5 mm for circular cross-sections without welded circumferential joints.
b) 2.0 mm for circular cross-sections with welded circumferential joints having reinforcement in the form of a backup plate.
c) 3.0 mm for circular cross-sections with unreinforced welded circumferential joints, and for all non-circular sections.

5—8.2.2 **Aluminum**

5—8.2.2.1 Aluminum for supports shall meet the requirements of CSA Standard S157-M, *Strength Design in Aluminum*.

Aluminum castings shall be cast from primary ingot melts, having known compositions and Aluminum Association alloy designations.

5—8.2.2.2 The minimum thickness of material for the chords of wrought aluminum truss supports shall be 4.5 mm, and for diagonal members and bracing the minimum thickness shall be 3 mm.

5—8.2.2.3 Pole type supports of wrought aluminum shall be of closed cross-section and the minimum thickness of material shall be 4.5 mm.

5—8.2.2.4 Aluminum bolts shall not be used in supports.

Camber

5—8.3

5—8.3.1 Horizontal support members shall be cambered to compensate for deflection due to unfactored dead loads.

5—8.3.2 In addition to the dead load camber specified in Clause 5—8.3.1, camber not less than L/1000 shall be provided for overhead sign supports, cantilevered sign supports and cantilevered traffic signal supports.

5—8.3.3 Tenons and similar members connecting luminaires to bracket arms shall be inclined upwards at a minimum slope of 3°, except where such inclination would adversely affect the lighting characteristics of the luminaire.

Connection Details

5—8.4

5—8.4.1 **Structural Steel**

5—8.4.1.1 All welds shall meet the requirements of Section 10.

5—8.4.1.2 Circumferential welds in tubular members shall be 100% penetration butt welds except as specified in Clause 5—8.4.1.3.

5—8.4.1.3 Connections of pole type members to base plates for luminaire supports not greater than 16 m in height, or for traffic signal supports, shall be 100% penetration butt welds or socket type connections with two fillet welds.

5—8.4.1.4 Lapped joints in tapered tubular members shall be of sufficient length to develop the full strength of the lapped members in flexure.

5—8.4.2 **Aluminum**

5—8.4.2.1 Fabrication and design details of aluminum members and components shall be in accordance with CSA Standard S157-M, *Strength Design in Aluminum*.

Anchor Bolts for Supports

5—8.5 Anchor bolts shall be embedded in concrete or in rock with sufficient embedment or mechanical anchorage to develop the full strength of the bolt before pullout. Anchor bolts fabricated from reinforcing steel shall not be used for supports.

Bridge Railings **5 — 9**

Limit States Design **5 — 9.1**
Railings shall be designed for the ultimate limit state in accordance with the requirements of Section 2. No serviceability limit state is prescribed for railings.

Railing Material **5 — 9.2**
Materials for combination or pedestrian railings shall be concrete, metal, timber, or a combination thereof. Metals with less than 20% tested elongation shall not be used. The tested elongation for cast metal shall be as obtained by testing cast test coupons in accordance with an approved method.

Railing Design **5 — 9.3**

5 — 9.3.1 **Combination Railings**

5 — 9.3.1.1 Unless otherwise approved, combination railings shall provide a continuous face of rail on the traffic side with the posts set back from the face of each rail a distance not less than 20 mm.

5 — 9.3.1.2 The thickness or depth of protrusions or depressions at rail joints shall not be greater than the wall thickness of the rail member nor more than 10 mm.

5 — 9.3.1.3 Structural continuity for flexure shall be provided in rail members.

5 — 9.3.2 **Pedestrian Railings**
Pedestrian railings shall provide a smooth continuous upper rail, free from sharp edges and rough seams. The thickness or depth of protrusions or depressions shall be no greater than the wall thickness of the rail member nor more than 10 mm, and their edges shall be finished in such a manner as to prevent injury to pedestrians. The maximum width of the top rail shall be 90 mm.

Application of Loads **5 — 9.4**

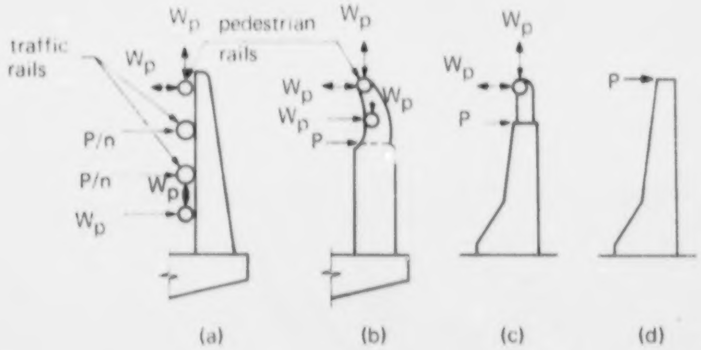
5 — 9.4.1 **Barrier Walls**
The traffic load on railings specified in Section 2 shall be applied as a concentrated load P at the top of the barrier wall as shown on Figure 5 — 9.4 (b), (c) and (d), and the response of the wall to this load shall be calculated in accordance with the analysis provisions for cantilever slabs in Section 7.

5 — 9.4.2 **Combination of Loads**
Vertical and horizontal loads prescribed for the design of railings shall be considered separately and in combination.

5 — 9.4.3 **Pedestrian Rails and Attachments**
The pedestrian load on railings specified in Section 2 shall be applied to pedestrian rails as a uniformly-distributed load W_p in each of the vertical directions. A uniformly-distributed load W_p shall be applied in each of the horizontal directions as shown on Figure 5 — 9.4(a), (b) and (c).

For the design of connections between posts and rails, an outward horizontal load of W_pL shall be applied to the rail. The connections shall also be designed for a load of $W_pL/4$ applied vertically downwards or vertically upwards, and in an inward horizontal direction.

Figure **Application of Loads** **5 — 9.4**



n = no. of traffic rails

5 — 9.4.4 **Traffic Rails and Attachments**

The traffic load on railings shall be applied as an outward horizontal load of P/n on each traffic rail, where P is the traffic load on railings specified in Section 2 and n is the number of traffic rails, as shown in Figure 5 — 9.4(a). The load P/n shall be applied as a concentrated load which may act at any point on the traffic rail.

The connections between traffic rails and posts shall be designed for an outward horizontal load of P/n, an inward horizontal load of P/4n, and an upward or downward vertical load of P/4n.

Rails located below or above the height range for traffic rails specified in Section 1 shall not be considered for the purposes of distributing the traffic load on railings and shall be designed as pedestrian rails.

5 — 9.4.5 **Posts Supporting Traffic Rails or Both Traffic Rails and Pedestrian Rails**

A concentrated load P/n shall be applied to each supporting post in an outward horizontal direction at the median level of each traffic rail. Simultaneously, a load of P/2nk shall be applied at the same points in the longitudinal direction of the rail, where k is the number of posts which support a single rail element between joints. The value of k shall not be taken as greater than 4.

Posts shall also be capable of resisting a concentrated load P/4n applied in an inward horizontal direction at the median level of each traffic rail and in an upward or downward direction.

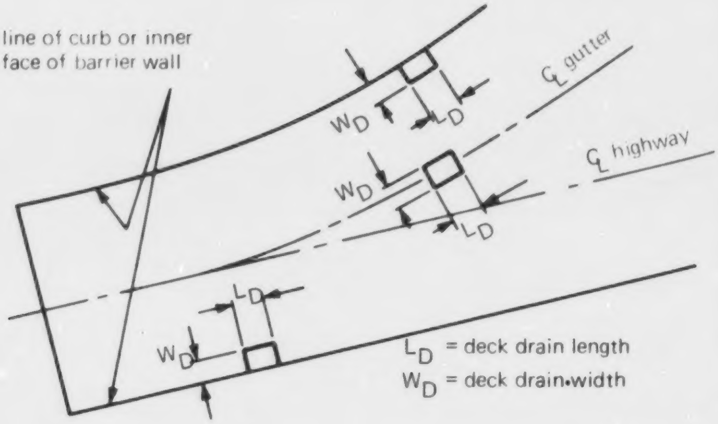
5 — 9.4.6 **Posts Supporting Only Pedestrian Rails**

A concentrated load W_pL, shall be applied to each supporting post in an outward horizontal direction at the median level of the uppermost pedestrian rail, or at a level of 1.5 m above the walkway surface, whichever is lower. The posts shall also be designed for a concentrated force $W_pL/4$ applied in an inward horizontal direction at the same level and also in an upward or downward direction.

Anchor Bolts for Posts **5 — 9.5**

At the ultimate limit state, the factored resistance of the anchor bolts for a railing post shall be not less than 1.2 times the factored resistance of the post.

Figure **Deck Drain Plan Dimensions** **5 — 10.1.1**



Bridge Deck Drains 5 — 10

Inlet Dimensions 5 — 10.1

5 — 10.1.1 The length and width of a deck drain for the purposes of this clause shall conform to its plan dimensions as defined in Figure 5 — 10.1.1. The deck drain length and the deck drain width shall each be not less than 200 mm.

5 — 10.1.2 The least dimension in plan of any opening of a deck drain inlet shall be not less than 40 mm and not greater than 75 mm. On highways on which cyclists are allowed, the dimensions of any opening measured parallel to direction of traffic or to curb line shall be not greater than 75 mm.

Deck Drain Placement 5 — 10.2

5 — 10.2.1 The tops of inlets shall be not less than 15 mm and not more than 25 mm below the general plane of the deck at the drain location and the deck surface around the drain shall be sloped towards the drain at a slope of 1 in 20 relative to the general plane.

5 — 10.2.2. Drains which are recessed into the faces of curbs or barrier walls shall not be less than 500 mm length.

Design of Drain Inlets 5 — 10.3

5 — 10.3.1 The design of steel inlets shall meet the requirements of Section 10.

5 — 10.3.2 Load bearing members of inlets shall be designed to support a uniformly-distributed load due to the heaviest wheel load including impact specified in Section 2, acting over a rectangle 250 mm in length by 600 mm in width.

Corrosion Protection 5 — 10.4

5 — 10.4.1 Steel deck drains including inlets shall be hot-dip galvanized after fabrication in accordance with CSA Standard G164-M1981, except as specified in Clause 5 — 10.4.2.

5 — 10.4.2 For drains to be used on structural steel bridges fabricated from Grade 350A steel supplied in accordance with Standard CAN3-G40.21-M81, *Structural Quality Steel*, the portions of the drains exposed below the deck shall be fabricated from the same steel and not galvanized. The portions of such drains embedded in concrete including the inlet shall be galvanized in accordance with CSA Standard G164-M1981, *Hot-Dipped Galvanizing of Irregularly-Shaped Articles*.

6 Substructures and Retaining Walls

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Scope 6—1

This section provides requirements for the design of substructures, concrete and bin-type retaining walls and manufactured and proprietary products.

Definitions 6—2

Basal Heave — The upward movement of the soil at the base of an excavation.

Bearing Capacity at Serviceability Limit States — The maximum load that can be applied to a soil or rock without settlements or other movements that may adversely affect the performance of a structure.

Bearing or Contact Pressure — The pressure applied by a shallow foundation to the soil or rock.

Bearing Surface — The contact surface between a shallow foundation and the soil or rock.

Bin-Type Retaining — A retaining wall constructed of continuously connected members forming a series of walls, usually rectangular or trapezoidal in plan, which are filled with granular material.

Capacity Modification Factor — A factor used to modify the factored capacity of a deep foundation unit when the factored capacity has been established from structural analysis.

Caisson — A wood, metal, or concrete casing sunk or constructed below ground or water level.

Deep Foundation — A foundation which derives its support by transferring loads to soil or rock at some depth below the structure, by end bearing, or by adhesion or friction or both.

Downdrag — The load transferred to a deep foundation unit by negative skin friction when the soil settles in relation to the unit.

Dynamic Methods of Analysis — The determination of the capacity of a deep foundation unit by wave analysis or dynamic driving formulae, applicable only in cohesionless soils and glacial tills.

Effective Height — The height measured in the vertical plane over which lateral earth pressure is assumed to act.

End Bearing Unit — A deep foundation unit which supports a load by its end or point bearing.

Expanded Base or Displacement Caisson Unit — A deep foundation unit formed by ramming concrete into the ground to form a bulb-shaped base.

Factored Axial Capacity of a Deep Foundation Unit — The axial capacity of a deep foundation unit, determined by load test or dynamic methods of analysis, and multiplied by an appropriate strength factor; or as determined by geotechnical formula using factored soil strength parameters.

Factored Bearing Capacity at Ultimate Limit States — The maximum load that can be applied without failure of a shallow foundation by breaking into the soil or rock, and computed using factored soil strength parameters.

Factored Soil Strength Parameters — The actual soil strength parameters multiplied by soil strength factors for cohesion and friction.

Friction Unit — A deep foundation unit which derives resistance by friction between its shaft and the surrounding soil.

Negative Skin Friction — Friction acting downward along the side of a deep foundation unit due to downdrag or applied uplift load.

Pile — A relatively slender deep foundation unit, wholly or partly embedded in the ground, installed by driving, drilling, augering, jetting or otherwise, and which derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip.

Pile Point or Driving Shoe — A reinforcement added to the tip of a pile to facilitate driving, to protect the pile end, and/or to improve its end-bearing capacity.

Reduction Factor for Inclined Load — A variable factor depending on the inclination of the resultant load, applied to reduce the capacity computed for a concentric load acting on a foundation.

Rock Point — A steel tip fitted to a pile to enable it to grip a sloping surface of bedrock.

Shallow Foundation — A foundation which derives its support by transferring load directly to the soil or rock at shallow depth.

Skin Friction — The resistance mobilized on the peripheral face of a deep foundation through cohesion and/or friction.

Soil Strength Factor — A factor applied to the soil strength parameters for cohesion or friction to obtain the factored soil strength parameters.

Strength Factor — A reduction factor applied to the capacity of a deep foundation unit when the capacity has been determined by means of a field test.

Substructure — A structural system through which the loads from a superstructure are transferred to the supporting soil or rock.

Subsurface Investigation — The appraisal of the general subsurface conditions by analysis of information obtained through geological surveys, *in situ* testing, boring, sampling, visual inspection, laboratory testing of samples of subsurface materials, and groundwater observations and measurements.

Notation

6—3

A	area of the underside of a shallow foundation, m^2.
A_p	cross-sectional area of a deep foundation unit of constant cross-section, m^2.
A'	effective bearing area of a shallow foundation, m^2.
A'_c	effective area within which deep foundation units share equally in compression, m^2.
A'_t	effective area within which deep foundation units share equally in tension, m^2.
B	width of a foundation, m.
B'	effective width of a foundation, m.
b	the average diameter of a round deep foundation unit, or the face-to-face dimension of octagonal, hexagonal, or square units, m.
C	axial strain wave velocity, m/s.
c	cohesion, kPa.
c_f	factored cohesion, kPa.
D	depth of a foundation below the final ground elevation, m.
D'	depth below the underside of a footing or pile cap measured vertically, m.
E	modulus of elasticity of material of a deep foundation unit, kPa.
E_h	average horizontal soil modulus along a deep foundation unit, MPa, $= k_s b$.
e_x	eccentricity of resultant load about the x-axis, m.
e_y	eccentricity of resultant load about the y-axis, m.
F_b	strength factor for a deep foundation unit where a load test has been used to determine Q_b.
F_c	soil strength factor for cohesion.
F_d	strength factor for a deep foundation unit where a dynamic method of analysis has been used to determine Q_d.
F_p	capacity modification factor for a deep foundation unit when the factored capacity of the unit has been established by means of structural analysis.
F_ϕ	soil strength factor for friction.
I_p	moment of inertia of a single deep foundation unit, m^4.
K_r	a non-dimensional parameter, $K_r = \dfrac{EI_p}{E_h\, l^4}$
k_s	modulus of horizontal subgrade reaction, $\dfrac{MN}{m^3}$
L	length of a foundation, m.
l	length in soil of a deep foundation unit, m.
L'	effective length of a foundation, m.
M_x	moment about the x-axis, kN·m.
M_y	moment about the y-axis, kN·m.
Q_b	unfactored axial capacity of a deep foundation unit determined by means of a load test, kN.
Q_d	unfactored axial capacity of a deep foundation unit determined by dynamic methods of analysis, kN.
Q_f	factored axial capacity of a deep foundation unit, at ultimate limit states, kN.
q_f	factored bearing capacity of a shallow foundation unit at ultimate limit states, kPa.
q_s	unfactored bearing capacity of a shallow foundation at serviceability limit states, Type II.
R_i	reduction factor for bearing capacity in the case of a shallow foundation; or reduction factor for load capacity in the case of a group of deep foundation units.
R_s	factored structural resistance of a deep foundation unit, kN.
α	inclination of load to the vertical, degrees.
α'	effective inclination of load to the vertical, degrees.
β'	effective inclination of group capacity of deep foundation units, degrees.
γ	unit weight of material, kN/m^3.
ϕ	angle of internal friction, degrees.
ϕ_f	modified angle of internal friction, degrees, obtained from the expression $\tan \phi_f = F\phi \tan \phi$.

Limit States 6—4

Serviceability Limit States 6—4.1

The serviceability limit states Type II to be considered for a substructure or retaining wall shall be those of total and differential movement. The substructure of a bridge or a retaining wall shall be proportioned so that the estimated total and differential movements are not greater than the movements that can be accommodated by the superstructure and the substructure. Unfactored soil strength parameters shall apply for all serviceability limit states Type II.

Ultimate Limit States 6—4.2

The ultimate limit states to be considered for a substructure or a retaining wall shall be those of:

a) failure of a shallow foundation by breaking into the underlying soil or rock and instability resulting in overturning or sliding; or

b) failure of a foundation through loss of support due to a geotechnical or structural failure.

Subsurface Evaluation and Parameters 6—5

Subsurface Evaluation 6—5.1

A subsurface investigation shall be carried out to provide the necessary information for the design and construction of foundations.

In all geotechnical-related aspects of substructure and retaining wall design, there shall be a dialogue between the structural designer and the geotechnical engineer, particularly during the formative stages of a design. The identification and classification of the soils and rocks encountered, and a description of their engineering and physical properties and groundwater conditions shall be established in order to evaluate subsurface conditions and probable foundation behaviour.

Soil Strength Parameters 6—5.2

The soil strength factors given in Table 6—5.2 shall be used to compute the factored soil strength parameters c_f and $\tan \phi_f$ from the following expressions:

a) Cohesion: $c_f = F_c c$

b) Friction: $\tan \phi_f = F_\phi \tan \phi$

where c and $\tan \phi$ are the representative mean values.

The factored soil strength parameters for cohesion, c_f, and for friction, $\tan \phi_f$, shall be used to compute factored earth pressure, factored bearing capacities for shallow foundations, and factored axial capacity of deep foundation units, all at the ultimate limit states. The unfactored soils strength parameters for cohesion, c, and for friction, $\tan \phi$, shall be used to compute settlements and other movements at the serviceability limit state Type II, and negative skin friction at the ultimate limit states.

Table 6—5.2 Soil Strength Factors, Ultimate Limit States

Cohesion	Stability and Earth Pressure	$F_c = 0.65$
Cohesion	Footings and Piles	$F_c = 0.50$
Friction	Stability and Earth Pressure	$F\phi = 0.80$
Friction	Footings and Piles	$F\phi = 0.80$

Bearing Capacity of Shallow Foundations on Rock and Unyielding Soil 6—5.3

6—5.3.1 Factored Bearing Capacity at Ultimate Limit States

At the ultimate limit state, the normally accepted values, without test, for factored bearing capacity for different types of generally sound bedrock and unyielding soil are as follows:

- Igneous and metamorphic rocks, e.g., granite — 10 000 kPa
- Limestone and sandstone — 3 000 kPa
- Shale — 1 500 kPa
- Hard or very dense glacial deposits — 1 000 kPa

Higher values may be used when substantiated by tests and experience. Lower values shall be used when the quality of the rock is weakened by joint fissures and other planes of weakness.

6—5.3.2 Bearing Capacity at Serviceability Limit States, Type II

Shallow foundations on bedrock or unyielding soil need not be checked for settlement since the loads required to produce detrimental settlement of the structure will be much larger than the recommended values for factored bearing capacity at ultimate limit states.

Loads and Load Effects — 6 — 6

Earth Pressure — 6 — 6.1

6 — 6.1.1 **General**

Earth pressure on a retaining structure shall be determined by considering the following:

- configuration, nature and drainage properties of the backfill material;
- displacement characteristics of the wall;
- interface conditions between the wall and the backfill;
- method of compaction of the backfill;
- water table.

6 — 6.1.2 **Computation of Earth Pressure**

6 — 6.1.2.1 Earth pressure shall be based on appropriate soil properties, and may be evaluated by using semi-empirical methods, method of equivalent fluid, or approved earth pressure theories. In the computation of earth pressure at ultimate limit states, the factored soil strength parameters shall be in accordance with Clause 6 — 5.2. and load factors shall be as specified in Section 2.

In the determination of earth pressure where drainage cannot be assured, the effects of water shall be considered.

6 — 6.1.2.2 Where the soil properties are not available, but the backfill surface is approximately level and specified as free-draining granular material and the retaining structure does not exceed 10 m in effective height, the following equivalent fluid pressures may be assumed.

a) At ultimate limit states
  i) active state — 8.0 kPa/m
  ii) at-rest condition — 12.0 kPa/m
  iii) passive state — 45.0 kPa/m
b) At serviceability limit states
  i) active state — 6.5 kPa/m
  ii) at-rest condition — 10.0 kPa/m
  iii) passive state — 60.0 kPa/m

When using the equivalent fluid pressures given for the active and at-rest condition, the load factors for earth pressure and their application shall be as specified in Section 2 and shall be applied to the equivalent fluid pressures shown.

When the passive state is in the nature of a resistance, a performance factor of 0.8 shall be applied to the equivalent fluid pressure prescribed for the ultimate limit state.

6 — 6.1.2.3 For cases other than those specified in Clause 6 — 6.1.2.2, or where the use of heavy vibratory compaction equipment is anticipated, the earth pressure shall be established by approved geotechnical methods.

6 — 6.1.2.4 **Surcharge Loads**

Where highway live loads can approach within a zone defined by a plane rising at 45° to the horizontal from the underside of the heel of a footing, an equivalent load due to a height of 600 mm of additional fill shall be assumed for the purpose of computing earth pressure. This load shall be assumed to act above the finished grade and over the entire length of the retaining structure. Where an adequately designed reinforced concrete approach slab supported by the bridge is provided, no equivalent surcharge load need be considered with the exception of wingwalls or retaining walls parallel to the span of the approach slab.

The effect of foundations or other loads placed in or on the backfill shall also be included.

Distribution of Wheel Loads — 6 — 6.2

6 — 6.2.1 The requirements of Clause 6 — 6.2 shall not apply to the design of soil-steel structures. The design of soil-steel structures shall be in accordance with Section 12.

6 — 6.2.2 When the depth of fill over a structure is 600 mm or more, wheel loads shall be uniformly distributed at the surface of the structure over a square, the sides of which are equal to 1.75 times the depth of fill.

6 — 6.2.3 When surcharges from several wheel loads overlap, the total load shall be considered as uniformly distributed over the areas defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the structure supporting the fill.

6 — 6.2.4 The effect of live load may be neglected when the depth of fill exceeds 2.5 m.

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6 — 6.2.5 When the depth of fill over a structure is less than 600 mm, wheel loads shall be considered to act as concentrated loads at the surface of the structure.

Backfill and Drainage **6 — 6.3** Free-draining granular material shall be specified as backfill to retaining structures. Provision shall be made to drain the backfill.

Water Pressure **6 — 6.4** High and low water levels shall be established for the life of the structure and shall be considered in the design.

Shallow Foundations **6 — 7**

General **6 — 7.1** The requirements of Clauses 6 — 7.2 to 6 — 7.3.4.5 shall apply to shallow foundations including single footings, wall footings and combined footings and mats. Combined footings and mats shall also meet the requirements of Clause 6 — 7.3.5.

Depth Criteria **6 — 7.2** The depth at which shallow foundations are to be founded shall be established considering:

a) the bearing capacity and compressibility characteristics of the soil or rock;
b) anticipated depth of scour as specified in Section 1;
c) possible ground movements;
d) future dredging or excavation adjacent to the foundations;
e) ground water level;
f) the extent of frost penetration;
g) the extent of seasonal volume changes of cohesive soils;
h) the proximity and depth of foundations of adjacent structures;
i) the overall slope stability.

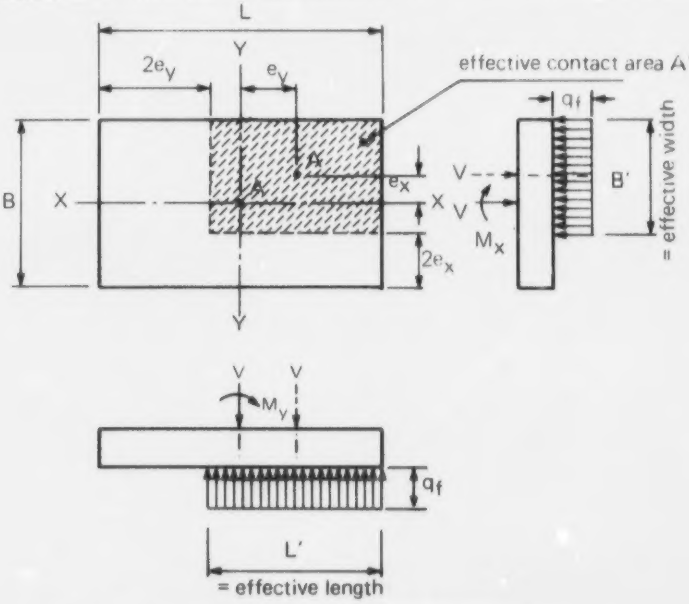
Design **6 — 7.3**

6 — 7.3.1 **General**

Shallow foundations shall be designed considering:

a) the bearing capacity of the supporting soil or rock, at serviceability and ultimate limit states;
b) the type of structure and superstructure, total and differential settlements and other movements that can be accommodated;
c) the duration and characteristics of loads; and
d) the depth of the foundation.

Figure **Effective Contact Area of a Shallow Foundation with Eccentric Load** **6 — 7.3.3.4**



effective area, A' = B'L'
effective width, B' = B - $2e_x$
effective length, L' = L - $2e_y$

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6 — 7.3.2 **Loads**

Shallow foundations shall be proportioned to resist all applicable loads as specified in Section 2.

6 — 7.3.3 **Bearing Capacity, Ultimate Limit States**

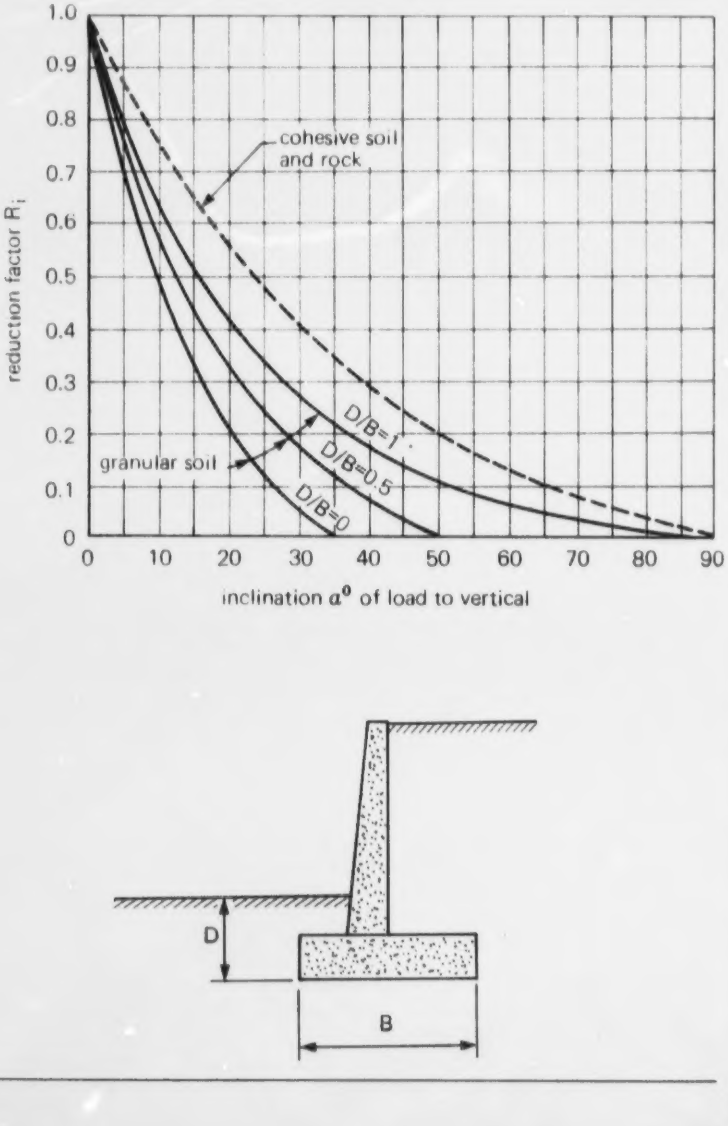
6 — 7.3.3.1 The factored bearing capacity at ultimate limit states shall be established by approved geotechnical methods using the factored soil strength parameters specified in Clause 6 — 5.2.

6 — 7.3.3.2 Consideration shall be given to sliding at the bearing surface at the ultimate limit states. For foundations on rock or soils which contain smooth surfaces, planes of weakness, joints or fissures, consideration shall be given to the possible reduction of both sliding resistance and bearing capacity.

6 — 7.3.3.3 For concentrically-loaded foundations, a uniform intensity contact pressure shall be assumed.

6 — 7.3.3.4 For the geotechnical analysis of eccentrically-loaded foundations, the contact pressure intensity at ultimate limit states shall be assumed to be uniformly distributed over the effective area A' as shown in Figure 6 — 7.3.3.4. If double eccentricity of the resultant load occurs, other uniform contact pressure distributions may be assumed, subject to approval.

Figure **Reduction for Inclination of Load Acting on a Shallow Foundation** **6 — 7.3.3.5**



6 — 7.3.3.5 Where the resultant load is inclined, the factored bearing capacity as determined in accordance with Clause 6 — 7.3.3.1, shall be reduced by multiplying by a factor R_i, which is to be obtained from Figure 6 — 7.3.3.5. The criteria for the reduction factor, R_i, are based on the type and homogeneous nature of the soil beneath the shallow foundation.

6 — 7.3.3.6 For the structural analysis of foundation footings, the uniform contact pressure distribution given in Clause 6 — 7.3.3.4 or a linear contact pressure distribution based on an elastic analysis shall be used, whichever produces the worst load effect. For the linear pressure distribution, the maximum contact pressure shall not be limited to the factored bearing capacity at ultimate limit states.

6 — 7.3.4 **Bearing Capacity, at Serviceability Limit States**

6 — 7.3.4.1 In the case of continuous structures or structures where total and differential settlement and other movements such as rotation can have an effect on the serviceability of a structure, the effect of these movements shall be taken into consideration.

6 — 7.3.4.2 For foundations on compressible strata, the initial settlements during which no drainage of water takes place from the soil mass within the influence of the loaded zone, as well as the subsequent consolidation settlement, shall be investigated.

6 — 7.3.4.3 The unfactored bearing capacity, and forces contributing to settlement, and other movements at serviceability limit states Type II shall be established by approved geotechnical methods using unfactored soil strength parameters. For concentrically- and eccentrically-loaded foundations, the contact pressure shall be of linear distribution.

6 — 7.3.4.4 In computing the time-dependent consolidation settlements in clay, only permanent loads shall be considered.

6 — 7.3.4.5 For computing total settlements in granular soils and the initial settlements in clay, both permanent and transitory loads shall be used.

6 — 7.3.5 **Combined Footings and Mats**

In both the geotechnical design and the structural design of combined footings and mats and in other cases where significant soil-strength interaction is likely, the magnitude and distribution of the contact pressures shall be determined considering the relative properties of the soil and the structure and using approved methods.

Deep Foundations 6 — 8

General **6 — 8.1** All loads shall be assumed to be transferred in their entirety by the deep foundation units to the underlying soil or rock, ignoring any contribution to bearing capacity from the footing.

Depth Criteria **6 — 8.2** The depth of the foundation units shall be established considering:

a) the bearing capacity and compressibility characteristics of the soil or rock;
b) permissible settlements of the structures;
c) anticipated depth of scour;
d) possible ground movements;
e) future dredging or excavation adjacent to the foundation;
f) proximity and depth of foundations of any adjacent structures, and
g) the probable location of the water table.

Design **6 — 8.3**

6 — 8.3.1 **Loads**

Deep foundation units shall be proportioned to resist all applicable loads as specified in Section 2.

6 — 8.3.2 **Determination of Capacity**

The axial capacity of a deep foundation unit at ultimate limit states shall be the factored axial capacity based on the ability of the soil or rock to support the unit as specified in Clause 6 — 8.3.3. The axial capacity of a deep foundation unit at serviceability limit states Type II shall be as specified in Clause 6 — 8.3.9. In all cases, the structural capacity shall be considered as specified in the appropriate Code section and noting Clause 6 — 8.3.11.

6 — 8.3.3 **Factored Axial Capacity**

6 — 8.3.3.1 Where a load test or dynamic methods of analysis are used, and the soil conditions are relatively homogenous, the axial factored capacity of a deep foundation unit shall be determined as follows:

a) Load test:

$$Q_f = F_b Q_b$$

where: F_b = 0.5 for routine testing, and
F_b = 0.6 for approved high level testing of single units or groups.

b) Dynamic methods of analysis:

$$Q_f = F_d Q_d$$

where: $F_d = 0.4$ for routine analysis, and
$F_d = 0.5$ for analysis based on parameters obtained from dynamic field measurements.

6 — 8.3.3.2 Where neither a load test nor dynamic methods of analysis have been used, the factored axial capacity of a deep foundation unit may be computed by an approved geotechnical formula and by applying factored soil strength parameters as specified in Clause 6 — 5.2. The factored capacity, Q_f, thus determined shall not exceed the value of $2.5EA_p/C$ nor the modified factored structural resistance, F_pR_s. The values of F_p and the source of R_s shall be as specified in Table 6 — 8.3.3.2.

Table **Capacity Modification Factors for Obtaining the Factored Axial Capacity of a Deep Foundation Unit, Q_f** **6 — 8.3.3.2**

	Capacity Modification Factor	**Factored Structural Resistance, R_s**
Type of Unit	**F_p**	**Source of R_s**
Precast Reinforced Concrete	0.4	Section 8
Cast-In-Place Concrete	0.4	Section 8
Expanded Base Concrete	0.4	Section 8
Prestressed Concrete	0.5	Section 9
Steel H-Section	0.5	Section 10
Unfilled Steel Pipe	0.5	Section 10
Concrete-Filled Steel Pipe	0.4	Section 10
Wood	0.4	Section 13

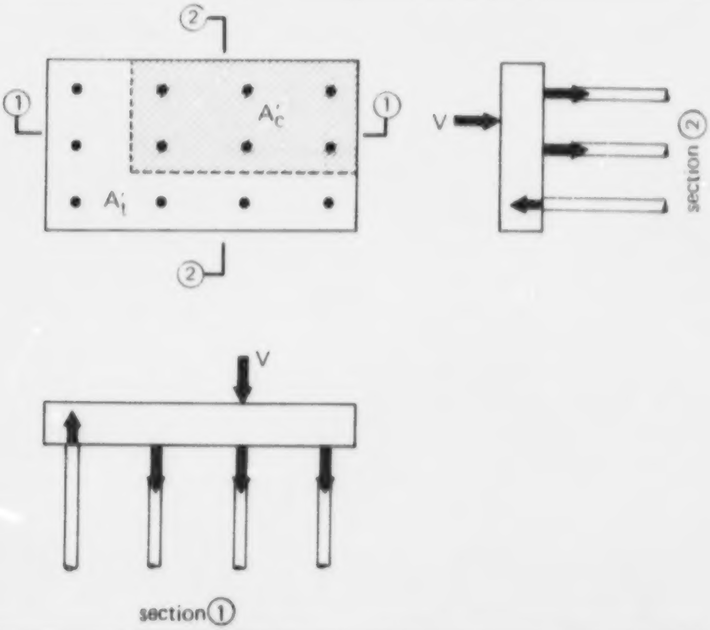
6 — 8.3.4 **Group Effect**

6 — 8.3.4.1 For centrally-loaded groups, equal load sharing on the deep foundation units shall be assumed.

6 — 8.3.4.2 For the geotechnical analysis of eccentrically loaded deep foundation units, the resistance provided at the ultimate limit states shall be assumed to be a uniform compression resistance over the effective area A_c' and a uniform tension resistance over the effective area A_t', not to exceed their respective ultimate limit states, as shown in Figure 6 — 8.3.4.2.

If double eccentricity of the resultant load occurs, other uniform resistance distributions may be assumed, subject to approval.

Figure **Effective Areas A_c' and A_t' of a Deep Foundation with an Eccentric Load** **6 — 8.3.4.2**



6—8.3.4.3 For deep foundation units where the resultant load is inclined to the vertical, the overall stability of the soil mass and the foundation units, shall be investigated. Where K, is more than 0.01, the vertical resistance of the group at the ultimate limit states, as determined in accordance with Clauses 6—8.3.4.1 and 6.8.3.4.2 shall be reduced by multiplying by a factor R, to be obtained from Figure 6—8.3.4.3, and the horizontal resistance of the group at the ultimate limit states shall be determined in accordance with Clause 6—8.3.8.

6—8.3.4.4 Where a group of deep foundation units at ultimate limit states contains inclined units, the effective angle of inclination to the vertical, α', shall be measured between the direction of the applied load and the average effective inclination of the units in the group, to determine the factor R, shown in Figure 6—8.3.4.3.

6—8.3.4.5 For the structural analysis of footings at ultimate limit states, the uniform distribution on the units given in Clauses 6—8.3.4.1, 6—8.3.4.2, 6—8.3.4.3 and 6—8.3.4.4 or a linear load distribution to the units based on an elastic analysis shall be used, whichever produces the worst load effect. For the linear distribution, the maximum load on a unit shall not be limited to the factored axial capacity of the unit at ultimate limit states.

6—8.3.4.6 For the geotechnical analysis of the foundation units and the structural analysis of the footing, the load distribution of the unfactored loads acting on the units shall be assumed to be linear at serviceability limit states, Type II, for concentrically- and eccentrically-loaded deep foundation units.

6—8.3.5 **Downdrag**

At ultimate limit states, the unfactored negative skin friction caused by downdrag on the deep foundation units shall be considered as equal to 1.25 times the unfactored soil strength. At the serviceability limit states, Type II, the negative skin friction on the deep foundation units shall be considered as equal to the unfactored soil strength.

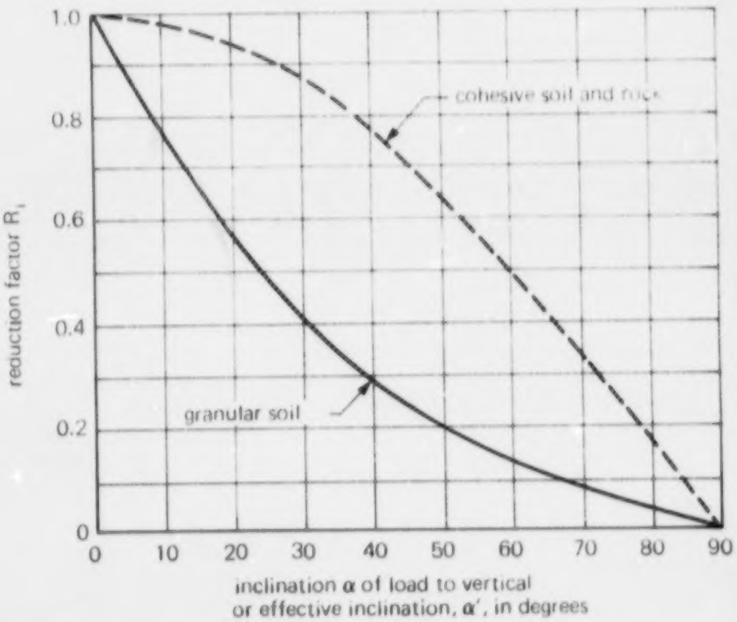
6—8.3.6 **Buoyancy**

The effect of hydrostatic pressure shall be considered in design at ultimate limit states and serviceability limit states, Type II.

6—8.3.7 **Uplift**

Deep foundation units may be considered to resist an uplift if provision is made for anchorage at the top, and if sufficient skin friction is developed. The uplift capacity of parallel-sided deep foundation units shall not exceed 50% of the factored axial skin friction capacity of the units computed by geotechnical formula, nor the weight of the material surrounding the units. In the case of tapered units, special consideration shall be given in determining their uplift capacity.

Figure **Reduction of Axial Capacity for Inclination of Load Acting on a Deep Foundation** **6—8.3.4.3**



6—8.3.8 Lateral Loads

In establishing the lateral resistance of a unit at ultimate limit states and serviceability limit states, Type II, consideration shall be given to the soil conditions surrounding the unit and the behaviour of the substructure. The lateral loads may be resisted by vertical and/or batter units, depending on the unit material, end restraint, the vertical load acting on the unit, and the soil restraint. The horizontal resistance of a battered unit shall be assumed to be proportional to the batter multiplied by the vertical load applied to the unit, or shall be determined by an approved method.

The lateral resistance of a group of deep foundation units is generally smaller than the sum of the lateral resistance of the individual units. This decrease shall be considered.

6—8.3.9 Bearing Capacity at Serviceability Limit States

In the case of continuous structures, or other structures where differential settlement and other movements such as rotation could have an effect on the serviceability of a structure, the effect of these movements shall be taken into consideration.

The bearing capacity, settlement, and other movements at serviceability limit states, Type II shall be established by approved geotechnical methods using unfactored soil strength parameters. For concentrically- and eccentrically-loaded foundations, the contact pressure shall be of linear distribution, as specified in Clause 6—8.3.4.6.

6—8.3.10 Spacing, Clearance, and Embedments

6—8.3.10.1 General

Spacing shall be measured from centre-to-centre of the units.

6—8.3.10.2 Minimum Spacing

The spacing between units at the underside of the footing or pile cap shall not be less than 2.50 b. At depth D' below the underside of the footing or pile cap, the minimum spacing shall be 2.50 b + 0.02 D', or 5.0 b, whichever is the lesser.

6—8.3.10.3 Clearance

Where the heads of units are incorporated in a concrete footing, the distance from the side of any unit to the nearest edge of the footing shall be not less than 250 mm.

6—8.3.10.4 Embedment in Footing

The heads of units shall project not less than 300 mm into the concrete after all material which has been damaged by driving has been removed. Where concrete and concrete-filled steel tube piles are provided with anchorage dowels extending into the footing concrete, the projection may be reduced to 100 mm.

6—8.3.11 Structural Resistance

6—8.3.11.1 The length of a unit permanently in contact with soil or rock shall be designed as a laterally-supported column.

6—8.3.11.2 The length of a unit in contact with air, water, or soft mud, including the length which may be exposed as a result of scour, shall be designed as a laterally-unsupported column.

6—8.3.11.3 The structural strength of a unit shall be sufficient to withstand load effects resulting from handling and driving.

6—8.3.11.4 For units installed in a corrosive environment, an allowance shall be made for a reduction in cross-section. The type and material of the unit shall be selected taking into account the effects of a corrosive environment.

6—8.3.11.5 Untreated Wood Piles

Untreated wood piles shall only be used under the following conditions:

a) in permanent construction where the pile cutoff is below the lowest established water table for the service life of the structure;
b) in a temporary construction when the designated service life of the piles is less than 5 years;
c) for protection fenders in waterways adjacent to bridge substructures.

6—8.3.12 Driving Points or Shoes

Driving points or shoes shall be used unless it is known from previous experience or special investigation that they are not necessary. The use of rock points shall be considered for short end-bearing units driven to hard rock surfaces, and for any units driven to an inclined hard rock surface where slippage of the tip could occur.

6—8.3.13 Splices

The strength and configuration of a splice shall not be such as to affect the performance of the unit. Wood piles shall not be spliced.

Inspection and Quality Control

6—8.4 The manufacture and installation of deep foundation units shall be inspected on a continuous basis by a qualified and experienced person, who is well acquainted with the design, to ensure:

a) that the site conditions are consistent with the design;
b) that the installation work is carried out in accordance with the design and good engineering practices;
c) that the deep foundation units are sound.

Piers, Abutments, and Retaining Walls

6—9

General

6—9.1 The design of piers, abutments, and retaining walls shall meet the requirements of Section 1 regarding location, clearances, protection and hydraulic design.

Loads

6—9.2

6—9.2.1 Piers, abutments, and retaining walls shall be designed for the applicable loads and load combinations specified in Section 2.

6—9.2.2 Earth pressure, surcharge loads, and water pressure shall be determined in accordance with Clause 6—6.

Foundations

6—9.3

6—9.3.1 Where piers, abutments, or retaining walls are underlain by deep strata of cohesive soils, the overall stability of the soil mass supporting the substructure or retaining wall shall be investigated.

6—9.3.2 Foundations placed on smooth or inclined bedrock surfaces shall be anchored into bedrock by means of keys, dowels, or sockets unless lateral stability can be assured by the overburden.

6—9.3.3 Where the loss of ground support or lateral restraint is anticipated, the foundation shall be protected by lowering its elevation or by the use of cutoff walls, permanent sheeting, slope protection, stream bed protection, training works or other appropriate means.

Structural Design

6—9.4 The structural design of piers, abutments, and retaining walls shall conform to the requirements of other applicable sections of this code.

Joints

6—9.5 Vertical contraction and expansion joints for concrete piers, abutments and retaining walls shall be provided as necessary to accommodate movements resulting from shrinkage and thermal changes.

Backfill and Drainage

6—9.6

6—9.6.1 Free-draining backfill shall be used within a wedge behind an abutment or retaining wall bounded by a plane rising at 60° to the horizontal, as shown in Figure 6—9.6.1.

6—9.6.2 The backfill material shall be drained by perforated pipes or weep holes.

Bin-Type Retaining Walls

6—10

General

6—10.1 The strength of a bin-wall, components, and connections shall conform to the requirements of other applicable sections of this Code.

Loads

6—10.2

6—10.2.1 Bin-walls, their components and connections shall be designed for the loads and load combinations specified in Section 2.

6—10.2.2 Earth pressure, surcharge loads, and water pressure shall be determined in accordance with Clause 6—6.

Design

6—10.3

6—10.3.1 Bin-walls shall be designed to resist the loads without failure by sliding or overturning.

6—10.3.2 Where a bin-wall is founded on deep strata of cohesive soils, the overall stability of the soil mass supporting the wall shall be investigated.

6—10.3.3 The effects of settlement and other movements on the bin-wall shall be investigated and provision made for them.

6—10.3.4 **Durability**
The possibility of corrosion shall be investigated and provision made for it.

Construction Requirements

6—10.4

6—10.4.1 Bin-walls shall be filled with free-draining granular material.

6—10.4.2 Free-draining material shall be placed behind the bin-wall to prevent surface water and groundwater from entering the bins. The minimum width of free-draining granular material behind the bins shall be 600 mm.

6 — 10.4.3 A perforated pipe of diameter not less than 150 mm shall be located in the granular material behind the wall at an elevation and grade suitable for draining the pipe, but at sufficient depth to drain the backfill effectively.

6 — 10.4.4 Provision shall be made to allow corner posts to settle sufficiently so that the loads are transferred to the underlying soil through the granular material contained within the bins.

6 — 10.4.5 The underside of a bin-wall shall be set at least 600 mm below the final grade at the front face of the wall. Where frost heave which may affect the performance of the structure is anticipated, the underside of the bin-wall shall be set below the frost line.

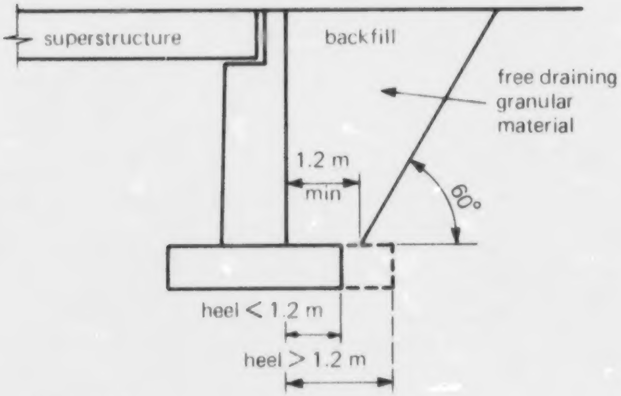
Sheet Walls 6 — 11

General 6 — 11.1

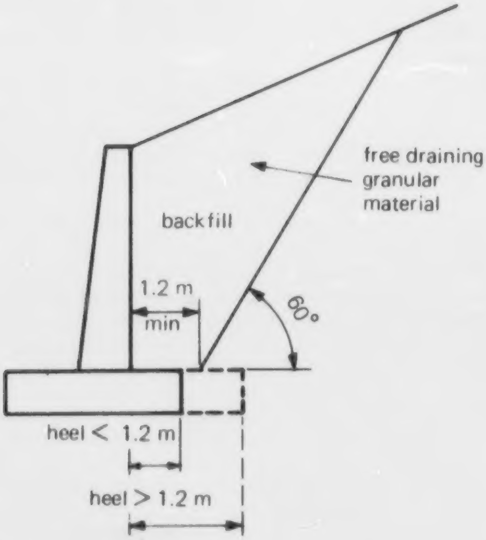
6 — 11.1.1 The requirements of Clause 6 — 11 shall apply to the design of the following types of sheet walls:

- cantilever walls
- anchored walls
- braced walls
- tied-back walls

Figure **Minimum Backfill Material Requirements for Abutments and Retaining Walls** **6 — 9.6.1**


superstructure
backfill
free draining granular material
1.2 m min
60°
heel < 1.2 m
heel > 1.2 m

Abutments


free draining granular material
backfill
1.2 m min
60°
heel < 1.2 m
heel > 1.2 m

Retaining walls

6 — 11.1.2 Sheet walls may be constructed of steel, concrete, wood or a combination of these materials.

6 — 11.1.3 For the design of sheet walls, the effects of time-dependent variables, including the effect on earth pressures of pressures due to frost action and water, shall be assessed. Corrosion shall also be assessed.

6 — 11.1.4 The design of a sheet wall shall meet the scour protection requirements specified in Section 1.

Loads

6 — 11.2

6 — 11.2.1 Sheet walls shall be designed for loads and load combinations specified in Section 2.

6 — 11.2.2 Earth pressure, surcharge loads, and water pressure shall be determined in accordance with Clause 6 — 6. In the determination of lateral loads due to earth pressure, the method and sequence of construction and the tolerable movements and deformation of the wall and soil shall be taken into account.

6 — 11.2.3 Closed wall systems shall be designed for the sum of earth and full groundwater pressure.

6 — 11.2.4 Water pressure may be neglected for open wall systems where uninterrupted seepage through the wall is provided.

Design

6 — 11.3

6 — 11.3.1 Sheet walls shall be driven to a depth sufficient to prevent basal heave.

6 — 11.3.2 Overall stability of the soil mass surrounding the sheet wall system shall be investigated.

6 — 11.3.3 The structural design, strength, and configuration of a sheet wall system shall conform to the requirements of other applicable sections of this code.

Manufactured and Proprietary Products

6 — 12

General

6 — 12.1 The requirements of Clause 6 — 12 shall apply to the design of substructures and retaining walls utilizing manufactured, fabricated, patented, and proprietary products.

Design

6 — 12.2 Manufactured, fabricated, patented, and proprietary products shall be designed and used in accordance with the manufacturer's or proprietor's recommendations and shall meet the requirements of this and other applicable sections of this code. In the absence of an acceptable theory, the design of a substructure or retaining wall utilizing manufactured, fabricated, patented or proprietary products may be based on an approved empirical method.

7 Concrete Deck Slabs

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Scope 7—1

This section prescribes provisions for the analysis and design of deck slabs of bridges. The slabs may or may not be prestressed. The provisions given apply generally to slabs which are solid of constant thickness, with skew angles up to 20°, and supported by girders, beams or other linear supports.

The use of an empirical method is emphasized, and is based on the significant in-plane forces which are developed in typical bridge deck slabs which satisfy certain conditions. In this method the slab design is reduced to a prescription of isotropic reinforcement. In cases where the empirical method is not applicable, a yield line method of analysis is preferred to an elastic method at ultimate limit states.

Definitions 7—2

Arching Action — development of compressive in-plane stresses in a slab due to restraint against lateral movement.

Deck Slab — a solid concrete slab, carrying and distributing wheel loads to the supporting members.

Isotropic Reinforcement — an arrangement of reinforcement in which the bars are orthogonal, and the reinforcement ratios in the two directions are equal.

Skew Angle — for slab panels which are in the form of a parallelogram, the angle formed by subtracting the acute angle of the parallelogram from 90°.

Yield Line — a line in a slab along which the ultimate moment is developed.

Notation 7—3

A_s	the area of non-prestressed tension reinforcement, mm².
A'	a coefficient given in Clause 7 — 10 and used in calculation of the bending moment in a cantilever slab due to a wheel load.
b	the width of compression face of member, mm.
C	the perpendicular distance from centre of wheel load to the line of support of a cantilever slab, m.
d	the effective depth of slab, being the distance from the extreme compressive fibre to the centroid of tension reinforcement, mm.
E	the width of slab over which a wheel load is distributed, m.
f_y	the specified yield stress of reinforcement, MPa.
f'_c	the specified compressive strength of concrete, MPa.
ℓ_1, ℓ_2	the dimensions of a skewed deck slab as shown in Figure 7 — 7.4.2.
M	the bending moment in a cantilever slab per unit length along a direction parallel to the support edge, kN·m/m.
M_y	the bending moment at a point x,y in a cantilever slab per unit length along a direction parallel to the supported edge, and producing rotation about the x-axis, kN·m/m.
P	the maximum factored wheel load as specified in Section 2, kN.
S	the effective span length of slab, m.
t	the slab thickness, mm.
x,y	a co-ordinate system referred to the load P, as shown in Figure 7 — 10.1.1, m.
X	the perpendicular distance, from centre of the area of the base of post, parapet or barrier wall, to a point under investigation, m.
ρ	the reinforcement ratio = $A_s/(bd)$.
ϕ	the angle of skew of a deck slab, as shown in Figure 7 — 7.4.2, degrees.

Limit States 7—4

Deck slabs shall be designed for the ultimate limit state of strength and the serviceability limit state of cracking. For slabs detailed in accordance with Clause 7 — 7.4, the serviceability limit state of cracking shall not be considered.

Span Lengths 7—5

For simple spans the span length shall be the distance centre to centre of supports but shall not exceed the clear span plus thickness of slab.

For a cantilever slab, the span length shall be taken as the distance from the end of the overhang to the supported edge, which is the centreline of the web in the case of a steel or wood girder, and to the external edge in the case of a concrete girder.

The following values of span lengths shall be used for continuous slabs over more than two supports:

- Slabs of constant thickness monolithic with beams or walls:
  S = clear span
- Slabs supported on steel stringers or concrete girders:
  S = distance between edges of flanges plus one half of the stringer or girder flange width.
- Slabs supported on wood stringers:
  S = clear span plus one half of the width of the stringer.

Methods of Analysis **7 — 6**

General **7 — 6.1**

Slabs which are proportioned by the provisions of Clause 7 — 7.4 need not be analysed. Other slabs shall be analysed in accordance with the provisions of Section 3. However, at the ultimate limit states, a yield line method is preferred to elastic methods. The yield line method may be used provided that the ratio of unfactored flexural resistance at a support to that at midspan in the same direction lies between 1.0 and 1.5.

Special Considerations **7 — 6.2**

The effects of the following shall be taken into consideration:

a) the skew angle of the slab, if greater than 20 degrees.
b) haunches,
c) the tendency of corners to lift under applied loads.

Design **7 — 7**

General **7 — 7.1**

The provisions of Sections 8 and 9 shall apply generally to the design of concrete deck slabs, except that Clause 7 — 7.4 where it applies, may be used in lieu of the provisions of those Sections. Materials shall conform to the provisions of Sections 8 and 9.

Crack Control **7 — 7.2**

The requirements of Clause 8 — 6.2.2 shall be met, except for slabs designed by the empirical method specified in Clause 7 — 7.4.

Concrete Protection for Reinforcement **7 — 7.3**

The provisions of Clause 8 — 9.3 shall apply. In the case of decks having an exposed concrete riding surface, the thickness of the deck, for design purposes, shall be reduced by not less than 10 mm as an allowance for wear.

Empirical Design of Composite Slabs **7 — 7.4**

7 — 7.4.1

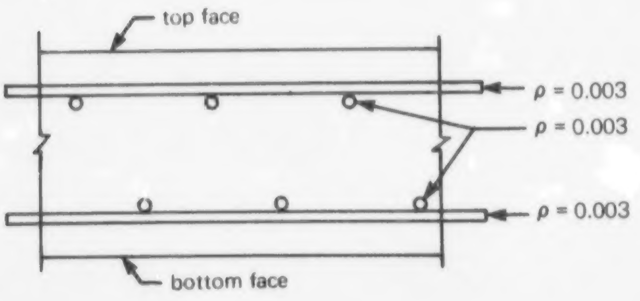
General

Deck slabs of composite beam slab systems shall be provided with a minimum of 0.3% isotropic reinforcement in each face as shown in Figure 7 — 7.4.1, if all the conditions of Clause 7 — 7.4.2 are met. The preferred orientation of reinforcement is shown in Figure 7 — 7.4.2. In cases where the transverse slab reinforcement is placed parallel to the abutment, the reinforcement shall be such that the minimum reinforcement ratio, expressed as a percentage, is not less than $0.3/\cos^2 \phi$. The design of cantilevered portions of the deck shall be in accordance with Clause 7 — 10. In cases where the cantilevered portion of a deck slab does not meet the requirements specified in Clause 7 — 7.4.2(a), the external slab panel shall be analysed in accordance with Clause 7 — 6.1.

Longitudinal slab reinforcement in negative moment regions of continuous composite girders shall be provided for, in accordance with the requirements of Sections 8, 9 and 10.

For steel box girders where the spacing of cross frames or diaphragms exceeds 8.0 m, the transverse moments and shear forces in the slab shall be investigated in accordance with Section 3 and provided for in addition to the requirements of this clause.

Figure **Isotropic Reinforcement Requirements** **7 — 7.4.1**



Slabs designed in accordance with Clause 7 — 7.4 shall be deemed to have met crack control requirements and to have adequate shear resistance.

For purposes of this clause the value of d used for design shall be taken as the average of the effective depths, at the midspan of the slab, in the two directions in which reinforcement is placed.

7 — 7.4.2 Conditions for Empirical Design

A mimimum of 0.3% isotropic reinforcement is permitted in deck slabs which satisfy the following requirements:

a) The span length of a slab panel perpendicular to the direction of traffic does not exceed 3.7 m, and the slab shall extend at least 1.0 m beyond the centreline of the external longitudinal supports of a panel. In the case of an external panel, a curb integral with the slab may be used instead of the 1.0 m overhang, provided that the combined cross-sectional area of slab and curb, beyond the centreline of the external girder, is not less than the cross-sectional area of one metre length of deck slab.

b) Span length to thickness ratio of the slab does not exceed 15. In skew slabs, the skew span shall be used in calculating this ratio.

c) For skew angles greater than 20°, the end portions of the deck slab shall be provided with 0.6% isotropic reinforcement as shown in Figure 7 — 7.4.2.

d) Slab thickness is not less than 225 mm, and spacing of the isotropic reinforcement bars in each face does not exceed 300 mm.

e) All cross frames or diaphragms shall extend throughout the cross section of the bridge between external girders, and the maximum spacing of such cross frames or diaphragms shall be as follows:
- Steel I and Box Girders: 8.0 m.
- Reinforced and prestressed concrete girders: at support lines.

f) Edge stiffening shall be provided in accordance with Clause 7 — 8.

Edge Stiffening 7 — 8

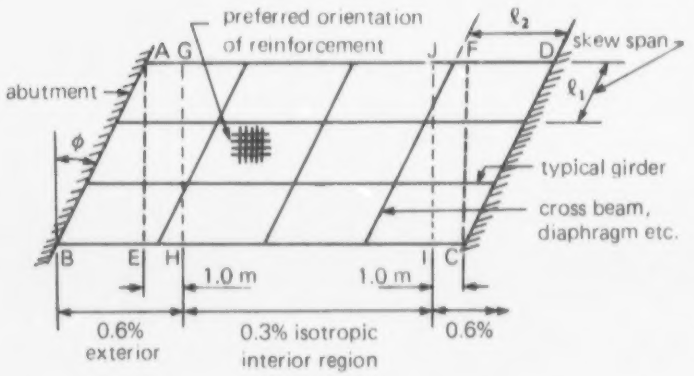
Longitudinal 7 — 8.1

Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. An edge beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, an integral reinforced section of slab and curb, or a continuous parapet or barrier wall. The edge beam shall be analysed in accordance with Appendix A3 — 1 of Section 3.

Transverse Unsupported Edges 7 — 8.2

Transverse edges at the ends of the bridge, and at intermediate points where the continuity of the slab is broken, shall be supported by diaphragms or other suitable means and shall be designed for the full effects of the wheel loads.

Figure **Skew Slab System** **7 — 7.4.2**



ϕ = angle of skew

ℓ_1/ℓ_2 = aspect ratio

$\phi \leq 20^\circ$: 0.3% isotropic (throughout)

Distribution Reinforcement 7—9

The provisions of this clause apply to cantilever slabs analysed by elastic methods, and do not apply to slabs designed in accordance with Clause 7—7.4 nor to slabs analysed by the yield line method.

Reinforcement shall be placed in all slabs transverse to the main steel reinforcement except in culverts or bridge slabs when the depth of fill over the slab exceeds 0.6 m.

The amount of distribution reinforcement shall be a percentage of the main reinforcement required for positive moment, as follows:

- With main reinforcement parallel to traffic:

  Percentage = $55/\sqrt{S}$, up to a maximum of 50%.
- With main reinforcement perpendicular to traffic:

  Percentage = $120/\sqrt{S}$, up to a maximum of 67%.

With main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement in the outer quarters of the span may be reduced by a maximum of 50%.

The reinforcement provided shall also conform to the requirements for temperature and shrinkage as specified in Clause 8—9.9.

Cantilever Slabs 7—10

Truck Loads 7—10.1

The effects of truck loads on cantilever slabs shall be assessed, using an elastic method or the yield line method. In the absence of more detailed calculation, the formulae given in Clauses 7—10.1.1, 7—10.1.2 may be used to calculate the bending moments in cantilever slabs, with or without edge stiffening.

7—10.1.1 Main Reinforcement Perpendicular to Traffic

The bending moment in cantilever slabs having reinforcement perpendicular to traffic shall be calculated in accordance with Clauses 7—10.1.1(a) and 7—10.1.1(b).

a) Spans not exceeding 1.2 m:

Moment per metre of slab, in direction parallel to traffic,

$$M = \frac{PC}{0.8C + 1.15} \text{ kN·m/m}$$

For a distance equal to S from an unsupported edge which is perpendicular to the direction of traffic, the amount of reinforcement provided shall be at least twice that required by the expression for M given above.

b) Spans exceeding 1.2 m:

For a cantilever slab with linearly varying thickness and having a span exceeding 1.2 m the bending moment, M_y, per metre due to a wheel load, P, shall be taken as:

$$M_y = \frac{PA'}{\pi} \quad \frac{1}{\cosh\left[\dfrac{A'x}{C-y}\right]}$$

where the coefficient A' is obtained from Figure 7—10.1.1, and the load is at least a distance, S, from an unsupported transverse edge. For a distance equal to S from an unsupported edge which is perpendicular to the direction of traffic, the amount of reinforcement provided shall be at least twice that required by the expression for M_y given above.

7—10.1.2 Main Reinforcement Parallel to Traffic

Moment per metre of slab, in direction perpendicular to traffic shall be taken as:

$$M = \frac{PC}{E} \text{ kN·m/m}$$

where E = 0.35C + 1.00, but shall not exceed 2.1 m.

Railing Loads 7—10.2

Railing loads shall be applied in accordance with Section 2. It shall be assumed that width, E, of the slab resisting railing loads is as specified below.

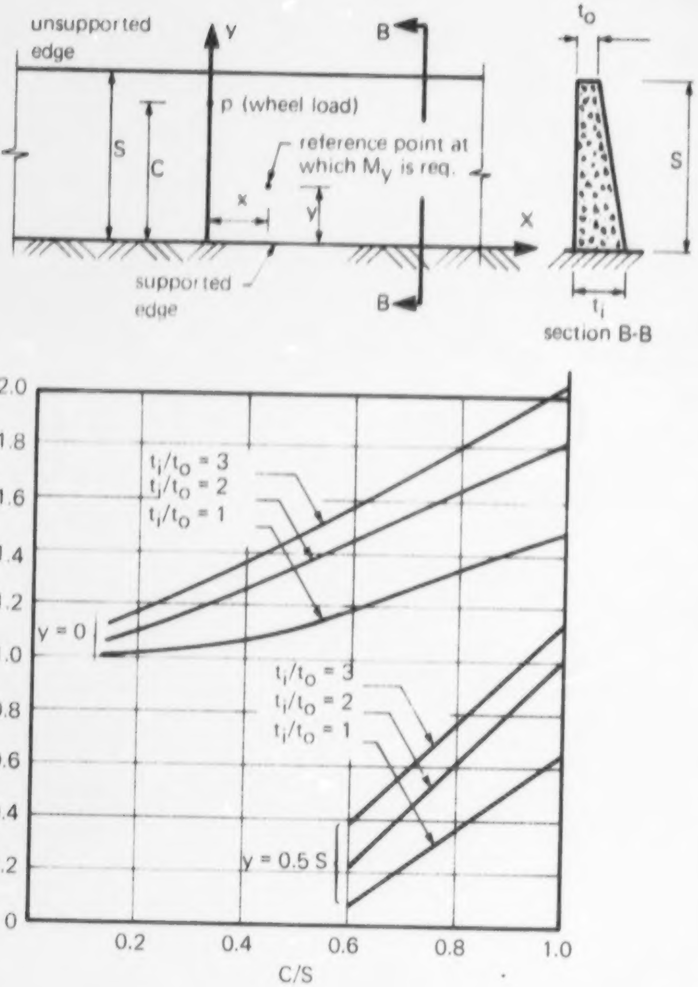
a) Slabs without parapet or barrier wall:

E = 0.8X + 1.15 in metres

b) Slabs with parapet or barrier wall:

E = 0.8X + 1.50 in metres

Figure **Values A' for Reference Points at y = 0 and y = 0.5S in Cantilever Spans Exceeding 1.2 m** **7 – 10.1.1**



8 Reinforced Concrete

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Reference Publications

Section 8 makes reference to the following publications

CSA Standards

CAN3-A23.1-M77	*Concrete Materials and Methods of Concrete Construction*
CAN3-A23.2-M77	*Methods of Test for Concrete*
CAN3-A23.3-M77	*Code for the Design of Concrete Structures for Buildings*
G30.12-M77	*Billet Steel Bars for Concrete Reinforcement*
G30.16-M77	*Weldable Low-Alloy Steel Deformed Bars for Concrete Reinforcement*
A251-M1982	*Qualification Code for Manufacturers of Architectural and Structural Precast Concrete*
W186-1970	*Welding of Reinforcing Bars in Reinforced Concrete Construction*
G30.3-1972(R1979)	*Cold-Drawn Steel Wire for Concrete Reinforcement*

Scope 8—1

The provisions of Section 8 apply to the design of bridge components, retaining walls, culverts, piles, and poles made of normal-density or low-density reinforced concrete. The requirements for the design of prestressed concrete members are given in Section 9.

Definitions 8—2

Balanced Condition — condition existing at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete at the extreme compression fibre reaches its assumed ultimate strain of 0.003.

Composite Construction — concrete members consisting of components which are constructed in separate placement but so interconnected that the components respond to loads as a unit.

Concrete Cover — the least distance between the surface of the reinforcing bars and the surface of the concrete.

Deep Flexural Members — flexural members with overall depth-to-span ratios greater than 0.4 for continuous spans and 0.8 for simple spans.

Development Length — the length of embedded reinforcement required to develop the specified strength of the reinforcement at a critical section.

Effective Area of Reinforcement — the area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Embedment Length — the length of embedded reinforcement provided beyond a critical section.

End Anchorage — a length of reinforcement, or a mechanical anchor or a hook, or combination thereof, beyond the point of zero stress in the reinforcement.

Equivalent Embedment Length — the length of embedded reinforcement which can develop the same strength as that which can be developed by a standard hook, or mechanical anchorage.

Low-Density Concrete — concrete containing structural low-density aggregate which conforms to the requirements of CSA Standard CAN3-A23.1 and which has a specified strength of not less than 20 MPa, with a mass density between 1400 and 2150 kg/m^3. In this section, low-density concrete without natural sand is referred to as "low-density concrete" and low-density concrete in which all fine aggregate consists of normal density sand is referred to as "semi low-density concrete".

Precast Members — concrete members produced in plants certified in accordance with CSA Standard A251, *Qualification Code for Manufacturers of Architectural and Structural Precast Concrete*.

Slab — a component having a width at least four times the effective depth.

Specified Strength of Concrete — the compressive strength of concrete assumed in the design and made and tested in accordance with the requirements of CSA Standard CAN3-A23.1, *Concrete Materials and Methods of Concrete Construction*, and CSA Standard CAN3-A23.2, *Methods of Test for Concrete*, 28 days after casting, or the compressive strength of concrete established by other approved means.

Spiral — continuously-wound reinforcement in the form of a cylindrical helix.

Splitting Tensile Strength — the tensile strength of concrete, 28 days after casting, determined by a splitting test made in accordance with CSA Standard CAN3-A23.2, *Methods of Test for Concrete*.

Stirrups or Ties — reinforcement used to resist shear and torsion in a structural member, typically bars, wires, or welded wire fabric, smooth or deformed, bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" is applied to those in compression members.

Yield Stress — the specified minimum stress at yield point of reinforcement.

Notation 8—3

A	the effective tension area of concrete surrounding the main reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, mm^2. When the main reinforcement consists of several bar sizes, the number of bars shall be computed as the total area divided by the area of the largest bar used.
A_b	the area of an individual bar, mm^2.
A_c	the area of core of a spirally-reinforced compression member measured out-to-out of spirals, mm^2; or the area enclosed by the perimeter of a section subject to torsion, mm^2.
A_{cr}	the area of the shear plane for shear-friction calculation, mm^2.
A_f	the area of reinforcement necessary to resist a moment due to factored load in a corbel or bracket, mm^2.

A_g	the gross cross-sectional area, mm^2.
A_h	the total area of stirrups or ties parallel to the main reinforcement in a corbel or bracket, mm^2.
A_n	the area of reinforcement necessary to resist the horizontal tension force in a corbel or bracket, mm^2.
A_ℓ	the total area of longitudinal reinforcement resisting torsion, mm^2.
A_o	the area enclosed by lines connecting the centres of the longitudinal bars in the corners of the closed stirrups, mm^2.
A_s	the area of tension reinforcement, mm^2.
A'_s	the area of compression reinforcement, mm^2.
A_{sf}	the area of reinforcement to develop the compressive resistance of overhanging flanges of flanged sections, mm^2.
A_{st}	the total area of longitudinal reinforcement in compression members, mm^2.
A_t	the area of one leg of a closed stirrup resisting torsion, mm^2.
A_v	the area of shear reinforcement within a distance s, mm^2.
A_{vf}	the area of shear-friction reinforcement, mm^2.
A_{vh}	the area of shear reinforcement parallel to main longitudinal reinforcement in deep shear spans, mm^2.
A_w	the area of an individual wire to be developed or spliced, mm^2.
A_1	the loaded area, mm^2.
A_2	the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2.
a	the depth of the equivalent rectangular stress block, mm, defined in Clause 8—7.2.7; or the shear span, mm.
a_b	the depth of the equivalent rectangular stress block for balanced conditions, mm.
b	the width of the compression face of the member, mm; or the band width for detailing reinforcement in a two-way rectangular footing, mm.
b_o	the periphery of the critical section for slabs and footings for punching shear, mm.
b_v	the width of the cross-section being investigated for interface shear, mm.
b_w	the web width, mm. For tapered webs, the average width or 1.2 times the minimum width, whichever is smaller; or the diameter of a circular section, mm.
C_m	a factor relating the actual moment diagram to an equivalent uniform moment diagram.
c	the distance from the extreme compression fibre to the neutral axis, mm.
d	the distance from the extreme compression fibre to the centroid of tension reinforcement, mm.
d'	the distance from the extreme compression fibre to the centroid of compression reinforcement, mm.
d''	the distance from the centroid of the gross section, neglecting the reinforcement, to the centroid of the tension reinforcement, mm.
d_b	the nominal diameter of a bar or wire, mm.
d_c	the thickness of concrete cover measured from the extreme tensile fibre to the centre of the bar located closest thereto, mm.
d_p	the diameter or depth of the cross-section of pile, mm.
E_c	the modulus of elasticity of concrete, MPa.
EI	the flexural stiffness of compression members, N·mm^2.
E_s	the modulus of elasticity of reinforcement, MPa.
e	the eccentricity of the applied load acting on a section at ultimate limit state and parallel to the axis measured from the centroid of the section, mm.
e_b	the eccentricity, M_b/P_b, at the balanced condition, mm.
f_b	the average bearing stress on the loaded area, MPa.
f'_c	the specified compressive strength of concrete, MPa.
f_{ct}	the average splitting tensile strength of low-density concrete, MPa.
f_h	the tensile stress developed by a standard hook, MPa.
f_r	the flexural tensile stress at cracking of concrete, MPa.
f_s	the tensile stress in reinforcement under loads at a serviceability limit state, MPa.
f'_s	the stress in the compression reinforcement at balanced conditions, MPa.
f_y	the specified yield stress of reinforcement, MPa.
h	the overall thickness of a member, mm.

h_f	the overall thickness of the flange of a flanged section, mm.
I_{cr}	the moment of inertia of the cracked section, transformed to concrete, mm^4.
I_e	the effective moment of inertia for computation of deflection, mm^4.
I_g	the moment of inertia of the gross concrete section about the centroid axis, neglecting the reinforcement, mm^4.
I_s	the moment of inertia of the reinforcement about the centroidal axis of the cross-section, mm^4.
k	an effective length factor for compression members
ℓ_a	the additional embedment length at support or at point of inflection, mm
ℓ_d	the development length, mm.
ℓ_e	the equivalent embedment length, mm.
ℓ_s	the distance from the point of zero shear to the centre of the support, mm.
ℓ_u	the unsupported length of a compression member, mm
ℓ_w	the total lengths of wire extending beyond outermost cross wires, for each pair of spliced wires, mm.
M	the moment acting on a section, due to factored loads, at ultimate limit state. N·mm
M_a	the maximum moment in a member at the stage for which deformation is being computed, N·mm.
M_b	the factored flexural resistance of a section at simultaneously assumed ultimate strain of concrete and yielding of tension reinforcement, N·mm.
M_c	the magnified moment used for proportioning slender compression members, N·mm
M_{cr}	the cracking moment $= f_r I_g / y_t$, N·mm.
M_t	the unfactored flexural resistance of a section in bending or combined bending and axial load, calculated using specified material properties and member sizes, N·mm
M_u	the factored flexural resistance of a section in bending or in combined bending and axial load, N·mm
M_{ux}	the factored flexural resistance of a section in the direction of the x-axis, N·mm
M_{uy}	the factored flexural resistance of a section in the direction of the y-axis, N·mm.
M_x	the component of moment due to factored load in the direction of the x-axis, N·mm
M_y	the component of moment due to factored load in the direction of the y-axis, N·mm
M_1	the value of the smaller end moment at ultimate limit state due to factored load acting on a compression member, positive if the member is bent in single curvature and negative if bent in double curvature, N·mm.
M_2	the value of the larger end moment at ultimate limit state due to factored load acting on a compression member, always positive, N·mm.
N	the axial force norm[illegible] cross-section, due to factored loads at the ultimate limit state occurring simultaneously with V, to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep, N, or the factored tensile force on a corbel or bracket acting simultaneously with V, N.
n	the modular ratio $= E_s/E_c$
P	the factored axial load acting on a section at the ultimate limit state, N.
P_b	the factored axial resistance of a section at simultaneously assumed ultimate strain of concrete and yielding of tension reinforcement, N.
P_c	the critical load, N.
P_m	the factored axial resistance of a section in compression with minimum eccentricity.
P_o	the factored axial resistance of a section in pure compression, N.
P_{ux}	the factored axial resistance corresponding to M_{ux}, N.
P_{uxy}	the factored axial resistance with biaxial loading, N.
P_{uy}	the factored axial resistance corresponding to M_{uy}, N.
p_c	the perimeter of a section, mm.
p_o	the perimeter of the area A_o, mm.
r	the radius of gyration of the gross cross-section of a compression member, mm.
s	the spacing of ties or stirrups, or spacing of shear or torsional reinforcement in a direction parallel to the longitudinal reinforcement, mm.

s_w	the spacing of wire to be developed or spliced, mm.
s_1	the spacing of horizontal stirrups in a direction perpendicular to the longitudinal reinforcement, mm.
T	the torsional moment at a section due to factored loads at the ultimate limit state, N·mm.
T_u	the factored torsional resistance, N·mm.
t_c	$(0.75\ A_c)/p_c$, mm.
t_o	$(0.75\ A_o)/p_o$, mm.
V	the shear force at a section due to factored loads at the ultimate limit state, N.
V_c	the shear resistance provided by concrete, N.
V_h	the shear resistance provided by shear reinforcement parallel to the longitudinal reinforcement in a deep shear span, N.
V_s	the shear resistance provided by shear reinforcement, N.
V_u	the factored shear resistance of a section, N.
v_b	the basic shear stress in beams, MPa.
v_c	the basic shear stress in slabs and footings, MPa.
v_{dh}	the interface shear stress at any section, MPa.
w	the mass density of concrete, kg/m^3.
w_m	the calculated maximum crack width at the level of reinforcement, mm.
y_t	the distance from the centroidal axis of a gross section neglecting the reinforcement, to the extreme fibre in tension, mm.
α	the angle between inclined shear reinforcement and the longitudinal axis of a member, degrees.
β	the ratio of the long side to the short side of a rectangular footing.
β_b	the ratio of area of bars cut off to total area of bars at a section.
β_c	the ratio of the long side to the short side of concentrated load or reaction area.
β_d	the ratio of maximum factored dead load moment to maximum factored total moment, always positive.
β_1	a factor defined in Clause 8—7.2.7.
γ	the angle between reinforcement and the shear plane, degrees.
δ	a moment magnification factor for compression members given in Clause 8—7.5.4.2(f).
λ	a factor to account for the effect of properties of low- and semi low-density concrete on shear strength, defined in Clause 8—8.2.7.
μ	the coefficient of friction.
ξ	a constant for computing tensile stress developed by a standard hook.
ρ	the ratio of tension reinforcement, A_s/bd.
ρ'	the ratio of compression reinforcement, A_s'/bd.
ρ_b	the reinforcement ratio producing balanced conditions.
ρ_f	the ratio of tension reinforcement of a flanged section, $A_{sf}/b_w d$.
ρ_{min}	the ratio of minimum tension reinforcement.
ρ_s	the ratio of volume of spiral reinforcement to total volume of core, out-to-out of spirals, of a spirally-reinforced compression member.
ρ_t	the ratio of longitudinal reinforcement in compression members, A_{st}/A_g.
ρ_w	the ratio of reinforcement, $A_s/b_w d$.
ϕ	a performance factor.

Materials 8—4

Concrete 8—4.1

8—4.1.1 Materials, methods of materials testing, and construction practices used in or for concrete in plain and reinforced concrete shall unless otherwise specified, conform to the following CSA Standards:

a) CSA Standard CAN3-A23.1, *Concrete Materials and Methods of Concrete Construction*, and

b) CSA Standard CAN3-A23.2, *Methods of Test for Concrete*.

8—4.1.2 The specified strength of the concrete, f_c', for which each part of the structure is designed, shall not be less than 20 MPa nor exceed 40 MPa and shall be shown on the plans. Higher strengths up to 55 MPa may be used if approved.

Reinforcing Steel 8—4.2

8—4.2.1 Reinforcing steel and methods of tests of reinforcing steel shall conform to the requirements of CSA Standard CAN3-A23.1, *Concrete Materials and Methods of Concrete Construction*.

8—4.2.2 All reinforcing bars, if they are to be bent, shall meet the bent test requirements for the respective grades as specified in CSA Standard G-30.12-M77, *Billet Steel Bars* *for Concrete Reinforcement*, except that reinforcing bars with a yield stress between 350 MPa and 400 MPa shall meet the bent test requirements specified for bars with a yield stress of 400 MPa.

8 — 4.2.3 The yield stress or grade of reinforcement used in design shall not exceed 400 MPa and shall be shown on the plans.

8 — 4.2.4 All bars for concrete reinforcement shall be deformed bars, except that plain bars may be used for spirals and wire mesh. Plain bars smaller than 10 mm in diameter may be used for stirrups or ties.

8 — 4.2.5 Plain reinforcing bars smaller than 10 mm in diameter which are used for ties or stirrups shall meet a bend test requirement of 180° around a pin of 4-bar diameter.

8 — 4.2.6 Reinforcement to be welded shall be Grade 400 in accordance with CSA Standard G30.16, *Weldable Low-Alloy Steel Deformed Bars for Concrete Reinforcement*.

General Considerations 8 — 5

General **8 — 5.1** The load effects shall be determined by elastic analysis.

Spans **8 — 5.2**

8 — 5.2.1 The span length of members which are not build integrally with their supports shall be taken as the clear span plus the depth of the member, but need not exceed the distance between centres of supports.

8 — 5.2.2 In the analysis of statically indeterminate structures, centre-to-centre distance between supports shall be used. Moments at faces of supports may be used to proportion members.

8 — 5.2.3 When fillets making an angle of 45° or more with the axis of a continuous or restrained member are build monolithically with the member and support, the face of support shall be taken at a section where the combined depth of the member and fillet is one-and-one-half times the depth of the member. Fillets shall not be considered as adding to the effective depth.

Stiffness Assumptions **8 — 5.3**

8 — 5.3.1 Any reasonable assumption may be adopted for computing the relative flexural and torsional stiffness of statically indeterminate structures, provided that the assumption is used consistently thoughout the analysis.

8 — 5.3.2 The effect of haunches shall be considered in analysis.

Material Properities **8 — 5.4**

8 — 5.4.1 The modulus of elasticity, E_c, for concrete shall be taken as $w^{1.5}\,0.043\sqrt{f'_c}$ for values of w between 1400 and 2500 kg/m³.

8 — 5.4.2 The modulus of elasticity of steel reinforcement shall be taken as 200 000 MPa.

Flexural Tensile Stress at Cracking **8 — 5.5** In the absence of test data, the flexural tensile stress at cracking, f_r, shall be taken as follows:

• Normal-density concrete	$0.60\sqrt{f'_c}$
• Semi low-density concrete	$0.50\sqrt{f'_c}$
• Low-density concrete	$0.45\sqrt{f'_c}$

Temperature, Shrinkage, and Creep **8 — 5.6** The effect of forces due to shrinkage, temperature changes, creep, elastic shortening, and settlement of supports shall be considered. Thermal, shrinkage, and creep coefficients shall be as specified in Section 2.

Lateral Supports for Flexural Members **8 — 5.7** Spacing of lateral supports for flexural members shall not exceed 50 times the least width, b, of the compression flange or face. Effects of lateral eccentricity of load shall be taken into account in determining the spacing of lateral supports.

Limit States 8 — 6

General **8 — 6.1** Structural components shall be proportioned to satisfy the requirements at serviceability limit states and ultimate limit states.

Serviceability Limit States **8 — 6.2**

8 — 6.2.1 **Serviceability Limit States, Type I**

8 — 6.2.1.1 The serviceability limit states Type I to be considered shall be those of fatigue and vibration.

8 — 6.2.1.2 **Fatigue**

a) **Reinforcement**
The provisions for fatigue of reinforcement shall be as specified in Section 11.

b) **Concrete**
The maximum compressive stress in concrete shall not exceed 0.50 f'_c at sections where stress reversals due to live load and dynamic load allowance occur. This stress limit shall not apply to concrete deck slabs, proportioned in accordance with Clause 7 — 7.4.

8 — 6.2.1.3 **Vibration**

The requirements of Section 2 shall be met.

8 — 6.2.2 **Serviceability Limit States, Type II**

8 — 6.2.2.1 The serviceability limit states Type II to be considered shall be those of cracking and deformations.

8 — 6.2.2.2 **Cracking**

Tensile reinforcement shall be distributed in zones of tension, so as to control crack widths. The cross-section shall be proportioned so that the crack width, w_m, computed from:

$$w_m = 11 \times 10^{-6} f_s (d_c A)^{0.33}$$

does not exceed 0.25 mm for severe exposure and 0.35 mm for other conditions.

8 — 6.2.2.3 **Deformations**

Deflections and rotations shall be considered in design. Attention shall be given to short- and long-term deformations which may affect the function of the structure, in accordance with Section 1.

Ultimate Limit States **8 — 6.3**

8 — 6.3.1 The ultimate limit states to be considered shall be those of strength and stability.

8 — 6.3.2 Structural components shall be proportioned to have factored resistances which are greater than or equal to the factored load effects as specified in Section 2.

Performance Factors **8 — 6.4**

8 — 6.4.1 The following values of performance factor, ϕ, shall be used:

Flexure, axial tension, or combined flexure and axial tension

a) Member with depth not exceeding 500 mm	—	0.80
b) Member with depth exceeding 500 mm	—	0.85
Shear and torsion	—	0.75
Compression with flexure	—	0.65
Bearing	—	0.65

The value of ϕ may be increased linearly from the value for compression with flexure to the value for flexure as the factored axial load, P, acting on a section at the ultimate limit state decreases from 0.10 $f'_c A_g$ or P_b, whichever is smaller, to zero.

A performance factor of 1.0 shall be used for development lengths specified in Clause 8 — 10.

Proportioning of Sections for Axial Load and Flexure 8 — 7

General **8 — 7.1** Clause 8 — 7 covers the proportioning of reinforced concrete members subject to flexure, axial load, or both.

Assumptions at Ultimate Limit States **8 — 7.2**

8 — 7.2.1 At the ultimate limit states, the calculation of factored resistance shall be based on the assumptions given in Clause 8 — 7.2 and on the applicable conditions of equilibrium and compatibility of strains.

8 — 7.2.2 Strains in the reinforcing steel and concrete shall be assumed to be directly proportional to the distance from the neutural axis.

8 — 7.2.3 The maximum usable strain at the extreme compression fibre shall be assumed to be equal to 0.003.

8 — 7.2.4 Stress in reinforcement below the specified yield stress, f_y, for the grade of steel used shall be taken as E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement shall be considered independent of strain and equal to f_y.

8 — 7.2.5 Tensile strength of the concrete shall be neglected in calculations of flexural and axial tensile resistances.

8 — 7.2.6 The relationship between the concrete compressive stress distribution and concrete strain may be assumed to be rectangular, trapezoidal, parabolic, or of any other shape which results in a prediction of strength in substantial agreement with the results of comprehensive tests.

8 — 7.2.7 The requirements of Clause 8 — 7.2.6 may be considered satisfied by an equivalent rectangular concrete stress distribution defined as follows:

> A concrete stress of 0.85 f'_c uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain. The distance, c, from the fibre of maximum compressive strain to the neutral axis is measured perpendicular to that axis. The factor β_1 shall be taken as 0.85 for specified strengths up to and including 30 MPa. For specified strengths over 30 MPa, β_1 shall be reduced continuously at a rate of 0.08 for each 10 MPa of strength in excess of 30 MPa, but β_1 shall not be less than 0.65.

Assumptions at Serviceability Limit States — 8 — 7.3

8 — 7.3.1 At the serviceability limit states, calculation of stress shall be based on the assumptions given in Clause 8 — 7.3 and on the applicable conditions of equilibrium and compatibility of strain.

8 — 7.3.2 The provisions of Clauses 8 — 7.2.2 and 8 — 7.2.5 shall apply except for the case of deep flexural members, where non-linear distribution of strain and lateral buckling shall be considered.

8 — 7.3.3 The stress in the concrete shall be assumed to be directly proportional to strain.

8 — 7.3.4 The modular ratio n may be taken as the nearest whole number, but shall not be less than 6.

8 — 7.3.5 An effective modular ratio of 2n shall be used to transform the compression reinforcing steel for stress computations for cases where the compressive stress is less than the specified yield stress.

Flexural Members — 8 — 7.4

8 — 7.4.1 **Reinforcing Steel Limitations**

8 — 7.4.1.1 For flexural members, the reinforcement ratio, ρ, provided shall not exceed 0.75 of the ratio, ρ_b, which would produce balanced conditions for the section under flexure without axial compression. For members with compression reinforcement, the portion of ρ_b contributed by compression reinforcement need not be reduced by the 0.75 factor.

8 — 7.4.1.2 At any section of a flexural member where tension reinforcement is required by analysis, the total amount of reinforcement provided shall be such that the calculated unfactored flexural resistance is not less than 1.25 times the cracking moment, where the cracking moment is defined as that which induces a tensile stress of 1.0 f_r in the concrete section.

8 — 7.4.1.3 The requirements of Clause 8 — 7.4.1.2 may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis.

8 — 7.4.2 **Factored Resistance**

8 — 7.4.2.1 The factored resistance, $M_u = \phi M_r$, shall be computed using the provisions of Clause 8 — 7.2. For typical sections, the expressions given in Clause 8 — 7.4.2 may be used.

8 — 7.4.2.2 **Rectangular Sections with Tension Reinforcement Only**

For rectangular sections the factored resistance may be computed by:

$$M_u = \phi\left[A_s f_y d\left(1 - 0.6\,\rho\frac{f_y}{f'_c}\right)\right]$$

or

$$= \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right]$$

where: $a = \dfrac{A_s f_y}{0.85\, f'_c b}$

The balanced reinforcement ratio, ρ_b, for rectangular sections with tension reinforcement only, is given by:

$$\rho_b = \left(\frac{0.85\,\beta_1 f'_c}{f_y}\right)\left(\frac{600}{600 + f_y}\right)$$

8 — 7.4.2.3 **Flanged Sections with Tension Reinforcement Only**

When the compression flange thickness is equal to or greater than the depth of the equivalent rectangular stress block, a, the factored resistance may be computed by the provisions given in Clause 8 — 7.4.2.2. When the compression flange thickness is less than a, the factored resistance may be computed by:

$$M_u = \phi\left[(A_s - A_{sf})\, f_y \left(d - \frac{a}{2}\right) + A_{sf} f_y\, (d - 0.5\, h_f)\right]$$

where $A_{sf} = \dfrac{0.85\, f'_c\, (b - b_w)\, h_f}{f_y}$

and $a = \dfrac{(A_s - A_{sf})\, f_y}{0.85\, f'_c\, b_w}$

The balanced reinforcement ratio, ρ_b, for flanged sections with tension reinforcement is given by:

$$\rho_b = \frac{b_w}{b}\left[\left(\frac{0.85\, \beta_1 f'_c}{f_y}\right)\left(\frac{600}{600 + f_y}\right) + \rho_f\right]$$

where $\rho_f = \dfrac{A_{sf}}{b_w d}$

8 — 7.4.2.4 **Rectangular Sections With Compression Reinforcement**

For a rectangular section in which:

$$\frac{(A_s - A'_s)}{bd} \geq 0.85\, \beta_1 \left(\frac{f'_c d'}{f_y d}\right)\left(\frac{600}{600 - f_y}\right)$$

the factored resistance may be computed by:

$$M_u = \phi\left[(A_s - A'_s)\, f_y \left(d - \frac{a}{2}\right) + A'_s f_y (d - d')\right]$$

where: $a = \dfrac{(A_s - A'_s)\, f_y}{0.85\, f'_c b}$

When the value of $(A_s - A'_s)/bd$ is less than the value given above, the factored resistance may be computed in accordance with Clause 8 — 7.4.2.2.

8 — 7.4.2.5 The balanced reinforcement ratio, ρ_b, for a rectangular section with compression reinforcement is given by:

$$\rho_b = \left(\frac{0.85\, \beta_1 f'_c}{f_y}\right)\left(\frac{600}{600 + f_y}\right) + \rho'\left(\frac{f'_s}{f_y}\right)$$

where: $f'_s = 600\left[1 - \left(\frac{d'}{d}\right)\left(\frac{600 + f_y}{600}\right)\right] \leq f_y$

Compression Members **8 — 7.5**

8 — 7.5.1 **General**

8 — 7.5.1.1 The proportioning of cross-sections subjected to combined flexure and axial load shall be based on the assumptions given in Clause 8 — 7.2. The effects of slenderness shall also be considered.

8 — 7.5.1.2 Members subject to compressive axial load shall be proportioned for the maximum factored moment that can accompany the factored axial load. The factored axial load, P, at a given eccentricity shall not exceed the value given in Clause 8 — 7.5.1.3. The maximum factored moment, M, shall be magnified for slenderness effects in accordance with Clause 8 — 7.5.4.

8 — 7.5.1.3 The factored resistance, P_m, of a compression member shall be computed as follows:

a) For a member with spiral reinforcement,

$$P_m = 0.85\, \phi\, [0.85\, f'_c\, (A_g - A_{st}) + f_y A_{st}]$$

b) For a member with tie reinforcement,

$$P_m = 0.80\, \phi\, [0.85\, f'_c\, (A_g - A_{st}) + f_y A_{st}]$$

8 — 7.5.1.4 The factored resistance of a member subject to axial load and flexure shall be computed in accordance with Clause 8 — 7.2 or approved methods.

8 — 7.5.2 **Reinforcement Limitations**

8 — 7.5.2.1 For members under combined flexure and compressive axial load, when the factored axial load, P, is less than the smaller of $0.10\, f'_c A_g$ or P_b, the reinforcement ratio, ρ, shall not exceed 0.75 of the ratio ρ_b which would produce balanced conditions for the section under flexure without axial load.

8 — 7.5.2.2 The longitudinal reinforcement ratio, ρ_t, for compression members shall not be less than 0.01 nor greater than 0.08, but preferably not greater than 0.04. The minimum number of longitudinal reinforcing bars shall be 6 for bars in a circular arrangement, and 4 for bars in a rectangular arrangement. The minimum size of bars shall be 15.

8 — 7.5.2.3 Where the proportioning of compression members is controlled by considerations other than applied loading, the provisions of Clause 8 — 7.5.2.2 shall not apply. The minimum area of longitudinal reinforcement for such members shall be that required for a member with a reduced effective area of concrete capable of resisting the factored loads or a longitudinal reinforcement ratio of 0.01, whichever is greater.

8 — 7.5.3 **Biaxial Loading**

In lieu of an analysis based on stress and strain compatibility for a loading condition of biaxial bending, non-circular members subjected to biaxial bending may be proportioned using the following approximate expressions:

a) When the factored resistance required is equal to or greater than $0.10 f'_c A_g$:

$$\frac{1}{P_{uxy}} = \frac{1}{P_{ux}} + \frac{1}{P_{uy}} - \frac{1}{P_o}$$

where: $P_o = \phi\,[0.85\,f'_c\,(A_g - A_{st}) + f_y\,A_{st}]$

b) When the factored resistance required is less than $0.10 f'_c A_g$:

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} \leq 1$$

8 — 7.5.4 **Slenderness Effects in Compression Members**

8 — 7.5.4.1 **General**

The proportioning of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall take into account the influence of axial loads and variable moment of inertia on member stiffness and moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads.

8 — 7.5.4.2 **Approximate Evaluation of Slenderness Effects**

In lieu of the requirements of Clause 8 — 7.5.4.1, the proportioning of compression members with slenderness ratios, $k\ell_u/r$, less than 100, may be based on the following approximate procedure:

a) The unsupported length, ℓ_u, of a compression member shall be taken as the clear distance between slabs, girders, or other members capable of providing lateral support for the compression member. Where haunches are present, the unsupported length shall be measured to the lower extremity of the haunch in the plane considered.

b) The radius of gyration, r, shall be taken equal to 0.30 times the overall dimensions in the direction in which stability is being considered for rectangular members, and 0.25 times the diameter for circular members. For other shapes, r shall be computed for the gross concrete section.

c) For members braced against side-sway, the effective length factor, k, shall be taken as 1.0, unless an analysis shows that a lower value may be used. For members not braced against side-sway, the effective length factor, k, shall be determined with due consideration of cracking and reinforcement on relative stiffness, and shall be taken as greater than 1.0.

d) For members braced against side-sway, the effects of slenderness may be neglected when the slenderness ratio, $k\ell_u/r$, is less than $34 - 12(M_1/M_2)$.

e) For members not braced against side-sway, the effects of slenderness may be neglected when $k\ell_u/r$ is less than 22.

f) Members shall be proportioned using the factored applied load, P, at the ultimate limit states, and a magnified moment, M_c, defined by:

$$M_c = \delta M_2$$

where: $$\delta = \frac{C_m}{1 - \dfrac{P}{\phi P_c}} \geq 1.0$$

and $$P_c = \frac{\pi^2 EI}{(K\ell_u)^2}$$

In lieu of a more precise calculation, EI may be taken as:

$$EI = \frac{0.2\,E_c I_g + E_s I_s}{1 + \beta_d}$$

or conservatively,

$$EI = \frac{0.4\,E_c I_g}{1 + \beta_d}$$

For members braced against side-sway, and without transverse loads between supports for the loading case under consideration, C_m may be taken as:

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4$$

For members braced against side-sway and with transverse loads between supports in the plane of loading, C_m may be determined by analysis. For all other cases C_m shall be taken as 1.0.

g) If computations show that there is no moment at both ends of a compression member or that computed end eccentricities are less than (15 + 0.03 h), M_2 shall be based on a minimum eccentricity of (15 + 0.03 h) about each principal axis. The minimum eccentricity shall not be applied about both axes simultaneously. The value of C_m shall be determined by either of the following:
   i) When computed end eccentricities are less than (15 + 0.03 h), computed moments may be used to evaluate M_1/M_2.
   ii) If computations show that there is essentially no moment at both ends of a compression member, the ratio M_1/M_2 shall be taken equal to one.

h) For bending about both principal axes, the moment about each axis shall be amplified by δ, computed from the corresponding conditions of restraint about that axis.

i) In frames which are not braced against side-sway, the flexural members shall be proportioned for the total magnified end moments of the compression members at the joint.

j) When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure and collectively resist the side-sway of the structure, the value of shall be computed for the member group from:

$$\delta = \frac{C_m}{1 - \dfrac{\Sigma P/\ell_u}{\Sigma \phi P_c/\ell_u}}$$

In proportioning each member in the group, δ shall be taken as the larger of:

i) the value computed for the group as a whole, or

ii) the value computed for the individual compression member, assuming its ends to be braced against side-sway.

Bearing **8 — 7.6**

8 — 7.6.1

The factored bearing resistance of concrete supports which are not laterally reinforced may be computed as $0.85\,\phi\,f'_c A_1$, except as provided below:

a) When the supporting surface is wider on all sides than the loaded area, the factored bearing resistance on the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

b) When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical and 2 horizontal.

8 — 7.6.2

When the factored applied load is in excess of that permitted in Clause 8 — 7.6.1, adequate provision shall be made to resist bursting and spalling forces.

Deformations **8 — 7.7**

8 — 7.7.1

Computation of Instantaneous Deformations

Deformations which occur immediately on application of load shall be computed by elastic methods. Unless values are obtained by a more comprehensive analysis, deformations shall be computed taking the modulus of elasticity for concrete in accordance with Clause 8 — 5.4 and taking the effective moment of inertia as follows:

$$I_e = I_g\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \leq I_g$$

where: $M_{cr} = \dfrac{f_r I_g}{y_t}$

8 — 7.7.2

Computation of Long-Term Deformations

In the absence of a detailed analysis, the additional long-term deformation of flexural members may be obtained by multiplying the instantaneous deformation by the following factor:

$$[2 - 1.2\,(A'_s/A_s)] \geq 0.60$$

Shear and Torsion 8—8

General 8—8.1

Clause 8—8 covers the proportioning of reinforced concrete members subjected to shear and torsion.

Shear 8—8.2

8—8.2.1 Shear reinforcement shall be provided in all flexural members, except for slabs and footings, and members where the shear at the ultimate limit state, V, is less than one-third of the factored resistance provided by the concrete, ϕV_c.

8—8.2.2 Where shear reinforcement is required, the area provided shall be not less than:

$$A_v = \frac{0.35\, b_w s}{f_y}$$

8—8.2.3 When shear reinforcement is required to satisfy Clause 8—8.2.4, minimum shear reinforcement in accordance with Clause 8—8.2.2 shall be provided beyond the last point where shear reinforcement is required for a distance equal to the effective depth of the member, d, or the distance to the point where V is one-third of ϕV_c, whichever is greater.

8—8.2.4 The factored shear resistance of members shall be computed from:

$$V_u = \phi (V_c + V_s)$$

where V_c is computed in accordance with Clauses 8—8.2.6 to 8—8.2.10, and V_s is computed in accordance with Clauses 8—8.2.11 to 8—8.2.14. Wherever applicable, the effect of inclined flexural compression in variable depth members may be included, and the effects of axial tension due to restrained shrinkage and creep shall be considered.

8—8.2.5 When the reaction in the direction of the applied shear introduces compression into the end region of a member, sections located at a distance less than d from the support may be proportioned for the values of V and V_c computed at a distance d from the face of the support. This Clause shall not apply to deep shear spans as specified in Clause 8—8.5.

8—8.2.6 The shear resistance V_c shall be taken as:

$$V_c = v_b b_w d$$

except as specified in Clauses 8—8.2.8 to 8—8.2.10.

8—8.2.7 The basic shear v_b used in calculating the shear resistance V_c shall be:

$$v_b = (0.07 + 10.0\rho_w)\, \lambda \sqrt{f'_c}$$

where: $0.08\, \lambda \sqrt{f'_c} \leq v_b \leq 0.19\, \lambda \sqrt{f'_c}$

and λ = 1.0 for normal-density concrete
= 0.85 for semi low-density concrete
= 0.75 for low-density concrete

Linear interpolation is permitted when partial sand replacement is used.

8—8.2.8 In one-way slabs or other components having a width of four or more times the effective depth and an effective depth of 300 mm or less, v_b shall be taken as not less than $0.17\, \lambda \sqrt{f'_c}$. For components having a width between two and four times the effective depth and an effective depth of 300 mm or less, v_b may be interpolated between the value given by Clause 8—8.2.7 and $0.17\, \lambda \sqrt{f'_c}$.

8—8.2.9 For members subjected to axial compressive load, the shear resistance shall be taken as:

$$V_c = v_b \left(1 + \frac{3N}{A_g f'_c}\right) b_w d$$

where N/A_g is in MPa.

8—8.2.10 For members subjected to axial tensile load, the shear resistance shall be taken as:

$$V_c = v_b \left(1 + \frac{2N}{\lambda A_g \sqrt{f'_c}}\right) b_w d$$

where N is negative in tension and N/A_g is in MPa.

8—8.2.11 When shear reinforcement perpendicular to the longitudinal axis of the member is used, V_s shall be taken as:

$$V_s = \frac{A_v f_y d}{s}$$

8—8.2.12 When inclined stirrups or bent bars are used as shear reinforcement, the following provisions shall apply:

a) For inclined stirrups,

$$V_s = \frac{(\sin \alpha + \cos \alpha)\, A_v f_y d}{s}$$

b) With shear reinforcement consisting of a single group of parallel bars, all bent at the same distance from the support,

$$V_s = A_v f_y \sin \alpha \leq 0.25\, b_w d\, \lambda \sqrt{f'_c}$$

c) With shear reinforcement consisting of groups of parallel bars bent at different distances from the support, V_s shall be taken as the lesser of the value computed from Clause 8 — 8.2.12(a) and $0.33\, b_w d \lambda \sqrt{f'_c}$

d) Only the centre three-fourths of the inclined portion of any bent longitudinal bar shall be considered effective for shear reinforcement.

e) Where more than one type of shear reinforcement is used to reinforce the same portion of the web, V_s shall be computed as the sum of the various types separately, but V_s shall not be taken as greater than three times the contribution of the stirrups alone.

8 — 8.2.13 When V_s exceeds $0.33\, b_w d\, \lambda \sqrt{f'_c}$, the maximum spacing of stirrups given in Clause 8 — 9.6.2.1 shall be reduced by one-half.

8 — 8.2.14 The value of V_s shall not exceed $0.66\, b_w d\, \lambda \sqrt{f'_c}$ and $(V_c + V_s)$ shall not exceed $0.20\, b_w d f'_c$

8 — 8.2.15 When a beam or beams frame monolithically into a girder, additional stirrups shall be provided in the girder. These additional stirrups shall be proportioned such that the factored resistance, $\phi A_s f_y$, is equal to or greater than the shear force transferred from the beams to the girder at the ultimate limit state. These stirrups shall be placed in the girder within a distance of one-half the depth of the beams on either side of the beams. This requirement may be waived if the shear force in the beam is less than $0.25\, b_w d\, \lambda \sqrt{f'_c}$, or if the lower face of the girder is more than one-half the depth of the beam below the bottom of the beam, or if the girder is supported on its lower face at the joint.

Torsion **8 — 8.3**

8 — 8.3.1 The effects of torsion may be neglected in the proportioning of members if the torsion at the ultimate limit state is less than

$$\phi\, 0.10\, A_c t_c \lambda \sqrt{f'_c}$$

8 — 8.3.2 If torsion in a member is required to maintain equilibrium in the structure, torsional reinforcement proportioned in accordance with Clauses 8 — 8.3.5 and 8 — 8.3.6 shall be provided. This reinforcement shall be in addition to that required to resist the other forces acting in combination with torsion.

8 — 8.3.3 If torsion in a member arises only because of the restraint of angular rotation by the adjacent members, then in proportioning the member, the effect of torsion may be neglected, provided that the following provisions are satisfied.

a) The moments and shears in the structure are computed assuming no torsional stiffness of the member.

b) Minimum torsional reinforcement is provided in accordance with Clause 8 — 8.3.4.

c) In those sections of adjacent members where moments will occur due to the torsional restraint provided by the member, minimum flexural reinforcement in accordance with Clause 8 — 7.4.1.2, anchored so as to achieve full development, is provided.

8 — 8.3.4 If torsional reinforcement is required, the minimum amount of closed stirrups and longitudinal reinforcement provided shall be such that

$$\sqrt{\left(\frac{A_t}{s}\right)\left(\frac{A_\ell}{p_o}\right)} \geq 0.30\, \frac{A_c t_c \lambda \sqrt{f'_c}}{A_o f_y}$$

Closed stirrups used to resist shear force and longitudinal steel used to resist flexure may be included as part of A_t/s and A_ℓ/p_o, respectively. Longitudinal reinforcement shall be anchored in accordance with Clause 8 — 10. A_ℓ/p_o shall not exceed $7(A_t/s)$.

8 — 8.3.5 The required area of closed stirrups shall be computed from:

$$A_t = \frac{Ts}{\phi\, 2 A_o f_y}$$

8 — 8.3.6 The required area of longitudinal reinforcement distributed symmetrically around the section shall be computed from:

$$A_\ell = \frac{T p_o}{\phi\, 2\, A_o f_y}$$

8 — 8.3.7 In the flexural compression zone of a member, the area of longitudinal torsional reinforcement required may be reduced by an amount equal to M/df_y where M is the moment acting at the section in combination with T.

8 — 8.3.8 The section shall be proportioned such that

$$\frac{T}{\phi 2 A_o t_o} + \frac{V}{\phi b_w d} \le 0.22 f'_c$$

8 — 8.3.9 In determining A_o and A_c for flanged sections, the width of overhanging flange used shall not exceed three times the thickness of the flange.

8 — 8.3.10 For hollow sections, t_c shall not exceed the actual wall thickness, and t_o shall not exceed the distance from the inside face of the wall to the centreline of the outside leg of the closed stirrup.

8 — 8.3.11 For open cross-sections, warping torsional resistance shall be taken into account.

Shear Friction **8 — 8.4**

8 — 8.4.1 The provisions of Clause 8 — 8.4 shall apply in cases where it is appropriate to consider shearing actions along a given plane such as at an existing or potential crack, the interface between dissimilar materials, or the interface between two concrete elements cast at different times.

8 — 8.4.2 A crack shall be assumed to occur along the shear path, and relative displacement shall be considered to be resisted by friction maintained by the shear friction reinforcement across the crack.

8 — 8.4.3 At least four reinforcing bars shall cross the shear plane, for which ρf_y shall not be less than 1.4 MPa. The value of ρ shall be computed as A_v, divided by the area of the shearing plane, A_{cr}.

8 — 8.4.4 The shear force V transmitted across the plane shall not exceed $0.20 \phi f'_c A_{cr}$, nor $5.5 \phi A_{cr}$.

8 — 8.4.5 When the shear friction reinforcement is perpendicular to the shear plane, the factored shear resistance shall be taken as

$$V_u = \phi \mu \rho A_{cr} f_y$$

8 — 8.4.6 Reinforcement inclined to the shear plane may be treated as shear friction reinforcement provided that its inclination is such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement and provided that the angle γ between the bars and the shear plane is greater than 30° and less than 90°. When the shear friction reinforcement is inclined to the shear plane, the shear resistance shall be based on

$$V_u \le \phi \rho A_{cr} f_y (\mu \sin \gamma + \cos \gamma)$$

8 — 8.4.7 The coefficient of friction shall be 1.4λ for concrete cast monolithically, 1.0λ for concrete placed against hardened concrete, and 0.7λ for concrete placed against structural steel.

8 — 8.4.8 Direct tension across the assumed shear plane shall be provided for by additional reinforcement. Direct compressive stresses may be taken as additive to the ρf_y value of the shear friction reinforcement.

8 — 8.4.9 The shear friction reinforcement shall be distributed across and along the assumed crack and shall be anchored on both sides of the assumed crack by embedment, hooks or by welding to special devices.

8 — 8.4.10 For the purpose of Clause 8 — 8.4, when concrete is placed against previously hardened concrete, the plans shall specify that the interface for shear transfer is clean, free of laitance and intentionally roughened.

8 — 8.4.11 When shear force is transferred between structural steel and concrete the plans shall specify that the steel shall be clean and without paint.

Deep Shear Spans **8 — 8.5**

8 — 8.5.1 The provisions of Clause 8 — 8.5 apply to:

a) Shear spans supporting a principal load with a/d less than 2.
b) Shear spans not supporting a principal load, or portions of beams supporting uniform loads in which the distance ℓ_s between the point of zero shear and the support is less than 3d.
c) The beams must be loaded on the top face and supported on the bottom face.
d) For beams loaded by members framing into the sides, the load may be assumed to be applied at the top of the supported member provided that reinforcement in accordance with Clause 8 — 8.2.15 is provided.

8 — 8.5.2 The principal load shall be taken as a load which causes at least 50% of the shear force at the ultimate limit state at the support of a given shear span. The shear span, a, shall be taken as the distance from the centre of the principal load to the centre of the support, but not more than 1.15 times the clear distance from the face of the load to the face of support. The value of ℓ_s shall be taken as the distance from the point of zero shear to the centre of the support, but not more than 1.15 times the clear distance from the point of zero shear to the face of the support.

8 — 8.5.3 The proportioning of deep shear spans for shear force shall be based on

$$V_u = \phi(V_c + V_s + V_h)$$

where V_c, V_s and V_h are calculated in accordance with Clauses 8 — 8.5.5 and 8 — 8.5.6.

8 — 8.5.4 The shear resistance $(V_s + V_h)$ shall not exceed $0.66\, b_w d \lambda \sqrt{f'_c}$ and the maximum shear resistance $(V_c + V_s + V_h)$ shall not exceed $0.2\, b_w d f'_c$ nor $5.5\, b_w d$.

8 — 8.5.5 For shear span, a, supporting a principal load, the requirements of Clause 8 — 8.5.3 shall be based on the following:

a) The shear force V shall be computed at a/2 and the shear reinforcement required at this section shall be used throughout the entire shear span.

b) The shear resistance, V_c, provided by the concrete shall be taken as

$$V_c = (2d/a)v_c b_w d \geq v_c b_w d$$

c) The shear resistance, V_s, provided by the vertical stirrups, shall be taken as

$$V_s = \frac{A_v f_y (a - d/2)}{s}$$

but not more than $(A_v f_y d)/s$ nor less than zero.

d) The shear resistance V_h provided by the stirrups, parallel to the main longitudinal reinforcement shall be taken as

$$V_h = A_{vh} f_y \frac{(3d/2 - a)}{s_h}$$

but not more than $(A_{vh} f_y d)/s_h$ nor less than zero.

8 — 8.5.6 The provisions of Clauses 8 — 8.5.3 to 8 — 8.5.5 shall apply to shear spans not supporting a principal load and to uniformly-loaded shear spans, except that the term "a" shall be taken as $(2\ell_s)/3$.

8 — 8.5.7 The area of shear reinforcement, A_v, perpendicular to the main reinforcement, shall not be less than $(0.35\, b_w s)/f_y$ and s shall not exceed d/4 or 450 mm, whichever is less.

8 — 8.5.8 For shear spans having a/d less than 2 or ℓ_s/d less than 3, the area of shear reinforcement parallel to longitudinal reinforcement, A_{vh}, shall not be less than $(0.35\, b_w s_h)/f_y$ and s_h shall not exceed d/3 or 450 mm, whichever is less.

8 — 8.5.9 The provisions of Clauses 8 — 8.5.7 and 8 — 8.5.8 may be waived for slabs and footings.

Brackets and Corbels **8 — 8.6**

8 — 8.6.1 The provisions of Clause 8 — 8.6 apply to brackets and corbels which have a shear span-to-depth ratio, a/d, of unity or less, and which are subjected to horizontal tensile force, N, equal to or less than the shear force V. The distance, d, shall be measured at a section adjacent to the face of the supports.

8 — 8.6.2 The section adjacent to the face of the support shall be proportioned to resist simultaneously a shear force V, a moment Va + N(h − d), and a horizontal tensile force, N, at the ultimate limit state.

8 — 8.6.3 The reinforcement A_{vf} required to resist the shear force shall be calculated in accordance with the provisions of Clause 8 — 8.4.

8 — 8.6.4 The reinforcement A_f required to resist the moment shall be calculated in accordance with the provisions of Clause 8 — 7.2.

8 — 8.6.5 The reinforcement A_n required to resist the tensile force N shall be taken as $N/(\phi f_y)$. The tensile force, N, shall not be taken as less than 0.20 V unless special provisions are made to avoid tensile forces due to restrained shrinkage and creep, so that the member is subjected to shear and moment only.

8 — 8.6.6 The area of main tension reinforcement A_s shall be taken as $(A_f + A_n)$ or $(0.67\, A_{vf} + A_n)$, whichever is greater.

8 — 8.6.7 The main tension reinforcement shall be anchored as close to the outer face of the corbel as cover requirements permit, either by welding a bar of equal diameter across the ends of the main reinforcing bars, or by some other means of positive anchorage.

8 — 8.6.8 Closed stirrups or ties parallel to the main tension reinforcement and having a total cross-sectional area A_h not less than $0.50\,(A_s - A_n)$ shall be uniformly distributed within two-thirds of the effective depth adjacent to the main tension reinforcement.

8 — 8.6.9 The value of $\rho = A_s/bd$ shall not be less than $(0.04\, f'_c)/f_y$.

8 — 8.6.10 The depth of the corbel or bracket at the outside edge of the bearing area shall not be less than one-half the effective depth of the corbel or bracket at the section adjacent to the face of the support.

Interface Shear for Composite Members

8 — 8.7

8 — 8.7.1 In a composite member, provision shall be made for the full transfer of shear forces at the interfaces of the separate components.

8 — 8.7.2 The proportioning for interface shear forces shall be based on:

$$V \leq \phi v_{dh} b_v d$$

in which d is for the entire composite section.

8 — 8.7.3 The value of v_{dh} shall not exceed the following:

a) 0.55 MPa when the minimum tie requirements of Clause 8 — 8.7.5 are satisfied and the contact surfaces are clean but not intentionally roughened.

b) 2.4 MPa when the minimum tie requirements of Clause 8 — 8.7.5 are satisfied and the contact surfaces are intentionally roughened.

8 — 8.7.4 If the provisions of Clauses 8 — 8.7.2 and 8 — 8.7.3 are not satisfied, proportioning for the interface shear force shall be based on the provisions of Clause 8 — 8.4.

8 — 8.7.5 Ties for interface shear may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. The tie area shall not be less than $(0.35\, b_v s)/f_y$, where f_y shall not exceed 400 MPa. The spacing, s, shall not exceed four times the least dimension of the supported element nor 600 mm.

8 — 8.7.6 Intentional roughness may be assumed only when the contact surface is roughened, clean, and free of laitance. This shall be specified in the plans.

Slabs and Footings

8 — 8.8

8 — 8.8.1 The provisions of Clause 8 — 8.8 apply to the proportioning of slabs and footings in the vicinity of concentrated loads and reactions where two-way action is present. Where a slab or footing acts as a beam, the provisions of Clause 8 — 8.2 also apply. For shear spans with a/d less than 2 and ℓ_s/d less than 3, the proportioning of slabs and footings shall be in accordance with Clause 8 — 8.5.

8 — 8.8.2 The factored shear resistance for two-way action shall be such that:

$$V \leq \phi V_c \text{ for slabs without shear reinforcement,}$$

and the greater of

$$V \leq \phi V_c$$

or $$V \leq \phi (0.50\, V_c + V_s) \text{ for slabs with shear reinforcement.}$$

The factored shear force, V, shall be that acting at the critical section.

8 — 8.8.3 The shear resistance of the concrete, V_c, shall be taken as:

$$V_c = v_c b_o d$$

where $$v_c = \left(1 + \frac{2}{\beta_c}\right) 0.15\, \lambda \sqrt{f'_c} \leq 0.30\, \lambda \sqrt{f'_c}$$

8 — 8.8.4 The critical section for calculation of the periphery, b_o, shall be perpendicular to the plane of the slab, and be located so that its length is a minimum. The periphery need not approach closer to the loaded area than lines located at a distance 0.50 d from that area and parallel to its boundaries. A circular shape may be replaced by a rectangular shape of equal perimeter.

8 — 8.8.5 The depth, d, shall be measured at the periphery, b_o, perpendicular to the plane of the slab, and shall be equal to the distance from the compression surface of the slab to the centroid of the two-way longitudinal tension reinforcement passing through the periphery b_o.

8 — 8.8.6 When stirrup or bent reinforcement is required, the value of V_s shall not exceed 1.5 V_c.

8 — 8.8.7 Shear reinforcement shall extend out from the loaded area in such a manner that a section joining the legs of the outermost stirrups or the outermost bend in the bent bars is equal to or greater than a distance h from the loaded area.

8 — 8.8.8 Shear resistance shall be investigated at the critical section defined in Clause 8 — 8.8.4 and at sections more distant from the loaded area. The outermost section checked shall be 0.50 d more distant from the loaded area than the section joining the legs of the outermost stirrups or the outermost bend in bent bars. Each critical section shall be located so that its periphery, b_o, is a minimum and does not approach closer to the loaded area than the critical section specified in Clause 8 — 8.8.4.

8 — 8.8.9 The shear resistance, V_c, for slabs with shear reinforcement as required by Clause 8 — 8.8.2 shall be calculated using the b_o and d values for the critical sections described in Clause 8 — 8.8.8. The value of V_s shall equal the vertical component of the specified strength of the shear reinforcement crossing a potential inclined crack extending at 45° from the intersection of the compression surface of the slab and a critical section located 0.50 d closer to the loaded area than the critical section under consideration.

8 — 8.8.10 When it is necessary to transfer a moment to the slab or footing in addition to the concentrated load, the reduction in shear resistance resulting from the transfer of moment shall be taken into account if the magnitude of the factored moment at the critical section exceeds 0.20 Vd.

Alternative Methods of Proportioning for Bending, Shear and Torsion. **8 — 8.9**

8 — 8.9.1 In lieu of the procedures described in Clauses 8 — 8.2, 8 — 8.3, 8 — 8.5 and 8 — 8.6, the resistance of a reinforced concrete member may be determined by an experimentally-verified rational procedure which takes into account:

a) the requirements of equilibrium for the section,
b) the requirements of geometric compatibility, and
c) the stress-strain characteristics of the steel and of the cracked concrete.

8 — 8.9.2 The minimum reinforcement requirements of Clauses 8 — 8.2.2, 8 — 8.3.4, and 8 — 7.4.1.2 shall be satisfied.

Details of Reinforcement **8 — 9**

Hooks and Bends **8 — 9.1**

8 — 9.1.1 A standard hook shall consist of:

a) a semicircular bend plus an extension of at least 4 bar diameters but not less than 60 mm at the free end of the bar; or
b) a 90° bend plus an extension of at least 12 bar diameters at the free end of the bar; or
c) for stirrup and tie anchorage only, either a 90° or a 135° bend plus an extension of at least 6 bar diameters, but not less than 60 mm at the free end of the bar.

Where non-standard hooks are used for components proportioned for earthquake forces, details of these hooks shall be shown on the plans.

8 — 9.1.2 **Minimum Bend Diameter**

8 — 9.1.2.1 The diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values given in Table 8 — 9.1.2.1, except that for Grade 300 reinforcement in sizes 10 to 35, inclusive, with turns not exceeding 180°, the minimum diameter shall be 5 bar diameters.

Table **Minimum Diameter of Bend** **8 — 9.1.2.1**

Bar Sizes	Minimum Diameter
10 through 25	6 bar diameters
30 through 35	8 bar diameters
45 and 55	10 bar diameters

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8 — 9.1.2.2 Inside diameter of bends for stirrups and ties shall not be less than 4 bar diameters for bar sizes 15 and smaller. For bars larger than size #15, the bends shall be in accordance with Table 8 — 9.1.2.1.

8 — 9.1.2.3 Inside diameter of bends in welded wire fabric, plain or deformed, for stirrups and ties shall not be less than 4 wire diameters for deformed wire larger than 7 mm and 2 wire diameters for all other wires. Bends with inside diameter of less than 8 wire diameters shall be not less than 4 wire diameters from the nearest welded intersection.

Spacing of Reinforcement **8 — 9.2**

8 — 9.2.1 For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than one and one-half times the nominal diameter of the bars, nor less than one and one-half times the maximum size of the coarse aggregate, nor less than 40 mm.

8 — 9.2.2 For precast concrete, the clear distance between parallel bars in a layer shall not be less than the nominal diameter of the bars, nor less than one and one-third times the maximum size of the coarse aggregate, nor less than 25 mm.

8—9.2.3 Where positive or negative reinforcement is placed in two or more layers, the bars in the upper layers shall be placed directly above those in the bottom layers with a clear distance between layers not less than 25 mm.

8—9.2.4 The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8—9.2.5 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle. Bars larger than size 35 shall be limited to 2 in any one bundle.

Bundled bars shall be enclosed within stirrups or ties. Individual bars in a bundle cut off within the span of a member shall terminate at different points with at least 40 bar diameters stagger. Where spacing limitations are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8—9.2.6 In walls and slabs, the principal reinforcement shall not be spaced further apart than 1.5 times the wall or slab thickness, nor more than 300 mm.

Concrete Protection for Reinforcement

8—9.3

8—9.3.1 The minimum concrete covers given in Table 8—9.3.1(a) shall be provided for reinforcement. The average concrete cover which allows for tolerances in placing reinforcement shall be used in proportioning the components. Unless otherwise approved, the tolerance shall be as given in Table 8—9.3.1(b). The average concrete covers and the permissible tolerance shall be shown on the plan.

8—9.3.2 For bundled bars, the minimum concrete cover shall be equal to the lesser of the diameter of a single bar of equivalent area and 50 mm, but not less than that given in Table 8—9.3.1(a). Bundled bars shall not be used within 100 mm of surfaces exposed to de-icing chemicals.

8—9.3.3 Exposed inserts and plates shall be protected from corrosion by approved methods.

Distribution of Reinforcement in Flexural Members

8—9.4

8—9.4.1 Principal tensile reinforcement shall be evenly distributed. The requirements of Clause 8—6.2.2.2 shall also apply.

Where girder flanges are in tension, the tensile reinforcement shall be distributed over the effective width specified in Section 3, or over a width equal to one-tenth of the girder span, whichever is less. If the effective width exceeds one-tenth of the span, additional longitudinal reinforcement shall be provided in the outer portions of the slab.

8—9.4.2 If the depth of the side face of a member exceeds 600 mm, longitudinal reinforcement having a total area of at least 10% of the principal tensile reinforcement shall be placed near the side faces of the member and distributed in the zone of flexural tension. The spacing of such reinforcement shall not exceed 300 mm nor the width of the web. Such reinforcement may be included in computing the flexural capacity only if a stress and strain compatibility analysis is made to determine stresses in the individual bars.

Lateral Reinforcement for Flexural Members

8—9.5

8—9.5.1 Where compression reinforcement for flexural members is required by analysis, the reinforcement shall be enclosed by ties or stirrups in accordance with Clause 8—9.8.

8—9.5.2 Closed ties or stirrups may be formed in one piece by overlapping standard stirrups or ties and hooks around a longitudinal bar, or formed in one or two pieces lap-spliced with a Class C splice in accordance with Clause 8—10.10.3.1.

Shear Reinforcement

8—9.6

8—9.6.1 **Types of Reinforcement**

8—9.6.1.1 Shear reinforcement may consist of:

a) stirrups perpendicular to the axis of the member or making an angle of 45° or more with the longitudinal tension reinforcement;

b) welded wire fabric with wires located perpendicular to the longitudinal axis of the member;

c) longitudinal bars with a bent portion making an angle of 30° or more with the longitudinal tensile reinforcement;

d) a combination of stirrups and bent bars;

e) spirals.

8 — 9.6.1.2 Shear reinforcement shall be anchored at both ends in accordance with Clause 8 — 10.9.

8 — 9.6.2 **Spacing of Shear Reinforcement**

8 — 9.6.2.1 Where shear reinforcement is required and is placed perpendicular to the axis of the member, the maximum spacing shall not exceed 0.50 d nor 600 mm.

8 — 9.6.2.2 Inclined stirrups and bent bars shall be spaced so that every 45° line, extending toward the reaction from the d/2 of the member to the longitudinal tension bars, is crossed by at least one line of shear reinforcement.

Table **Minimum Concrete Cover** **8 — 9.3.1(a)**

Type of Member or Expo[illegible]ondition	Cast-In-Place Concrete, mm	Precast Concrete, mm
(a) Concrete not submerged or exposed to earth or de-icing chemicals		
– Principal reinforcement	40	25
– Stirrups, ties and spirals	30	25
(b) Concrete cast against and permanently exposed to earth	75	—
(c) Concrete submerged or exposed to earth		
– Principal reinforcement	50	35
– Stirrups, ties and spirals	40	25
(d) Deck slabs, medians, curbs and sidewalks		
– Top	50	35
– Soffit, for slabs ≤ 300 mm thick	30	25
– Soffit, for slabs > 300 mm thick	40	30
(e) Precast beams	—	25
(f) Barrier and parapet walls	50	35
(g) Concrete, other than components specifically covered in (a) to (f), exposed to surface run-off or spray containing de-icing chemicals	60	45

Table **Placing Tolerances for Reinforcement** **8 — 9.3.1(b)**

Type of Member or Exposure Condition	Cast-In-Place Concrete, mm	Precast Concrete, mm
(a) Concrete not submerged or exposed to earth or de-icing chemicals		
– Principal reinforcement	±20	±15
– Stirrups, ties and spirals	±20	±15
Interior surfaces of a closed cell	±10	± 5
(b) Concrete cast against and permanently exposed to earth	±25	
(c) Concrete submerged or exposed to earth		
– Principal reinforcement	±20	±15
– Stirrups, ties and spirals	±20	±15
(d) Deck slabs, medians, curbs and sidewalks		
– Top	±20	±15
– Soffit, for slabs ≤ 300 mm thick	±10	±10
– Soffit, for slabs > 300 mm thick	±10	±10
(e) Precast Beams		+ 5 – 3
(f) Barrier and parapet walls	±20	±15
(g) Concrete, other than components specifically covered in (a) to (f), exposed to surface run-off or spray containing de-icing chemicals	±20	±15

Torsional Reinforcement **8—9.7**

8—9.7.1 Torsional reinforcement shall consist of both closed stirrups perpendicular to the axis of the member, and longitudinal bars.

8—9.7.2 The spacing of the closed stirrups shall not exceed 0.13 p_c nor 300 mm.

8—9.7.3 The spacing of longitudinal bars distributed around the perimeter of the stirrups shall not exceed 300 mm.

8—9.7.4 At least one longitudinal bar having a diameter of not less than 0.06s and being not smaller than size 10 shall be placed inside each corner of the closed stirrups; these corner bars shall be anchored in accordance with Clause 8—10.

8—9.7.5 Anchored torsional reinforcement shall be provided for at least a distance 0.50 p_c beyond a point where this reinforcement is not theoretically required.

8—9.7.6 Closed stirrups shall be anchored with 135° standard hooks.

Lateral Reinforcement for Compression Members **8—9.8**

8—9.8.1 Spiral reinforcement for compression members shall consist of evenly-spaced continuous spirals held firmly in place by attachment to the longitudinal reinforcement and kept true-to-line by vertical spacers.

8—9.8.2 The spirals shall be of such size as to permit handling and placing without being distorted from the specified dimensions.

8—9.8.3 Spiral reinforcement shall have a diameter of at least 10 mm and may be plain or deformed.

8—9.8.4 Anchorage of spiral reinforcement shall be provided by one and one-half extra turns of spiral bar or wire at each end of the spiral unit.

8—9.8.5 Splices in spiral bars or wires shall be tension lap splices of 48 diameters minimum but not less than 300 mm, or welds in accordance with Clause 8—10.10.2. The clear spacing between spirals shall not exceed 75 mm, nor be less than 25 mm or one and one-third times the maximum size of coarse aggregate.

8—9.8.6 Spiral reinforcement shall extend from the footing or other support to the level of the lowest horizontal reinforcement in the members supported above.

8—9.8.7 The ratio of spiral reinforcement, ρ_s, shall not be less than the value given by:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}$$

8—9.8.8 All bars for tied compression members shall be enclosed by lateral ties, which shall be at least size 10 for longitudinal bars size 30 or smaller, and at least size 15 for longitudinal bars over size 30 and for bundled longitudinal bars. The spacing of the ties shall not exceed 16 longitudinal bar diameters nor the least dimension of the member, nor 300 mm, except that this spacing may be increased for compression members which have a larger cross-section than required by conditions of loading, in which case the maximum spacing shall not exceed 450 mm.

8—9.8.9 The ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°, and no bar shall be farther than 300 mm clear on either side from a laterally-supported bar.

8—9.8.10 Ties shall be located vertically not more than half a tie spacing above the footing or from other support, and not more than half a tie spacing below the lowest horizontal reinforcement in the members supported above. Welded wire fabric of equivalent area may be used. Where the bars are located around the periphery of a circle, a complete circular tie may be used provided that the splices adjacent to ties are staggered.

Shrinkage and Temperature Reinforcement **8—9.9**

8—9.9.1 Reinforcement for shrinkage and temperature normal to the principal reinforcement shall be provided in structural components and slabs where the principal reinforcement extends in one direction only. At all sections where it is required, such reinforcement shall be developed for its specified yield stress in accordance with Clause 8—10.

8 — 9.9.2 For components up to 250 mm in thickness, the ratio of shrinkage and temperature reinforcement area to gross concrete area shall not be less than:

a) 0.0020; for deformed bars having a yield stress less than 400 MPa.

b) 0.0018; for deformed bars having a yield stress of at least 400 MPa or welded wire fabric, deformed or plain.

8 — 9.9.3 For components greater than 250 mm in thickness, the minimum area of reinforcement provided in each face shall be 250 mm²/m in each direction.

8 — 9.9.4 The spacing of reinforcement shall not exceed three times the thickness of the component nor 300 mm.

8 — 9.9.5 Temperature and shrinkage reinforcement for vertical faces of beams shall be in accordance with Clause 8 — 9.4.2.

Development and Splicing of Reinforcement — 8 — 10

Development Requirements — 8 — 10.1

8 — 10.1.1 **General**

8 — 10.1.1.1 The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section either by embedment length or by end anchorage or by a combination thereof. For bars in tension, hooks may be used in developing the bars.

8 — 10.1.1.2 Tension reinforcement may be anchored in the compression zone or by bending it and making it continuous with the reinforcement on the opposite face of the member.

8 — 10.1.1.3 Reinforcement shall extend beyond the point at which it is theoretically no longer required to resist flexure, for a distance equal to the effective depth of the member, or 12 d_b or 1/16 of the clear span, whichever is greatest, except at the supports of simple spans and at the free end of cantilevers.

8 — 10.1.1.4 Reinforcement shall have an embedment length not less than the development length, ℓ_d, beyond the point where bent or terminated tensile reinforcement is no longer required to resist flexure.

8 — 10.1.1.5 Flexural reinforcement located within the width of a member and used to compute the shear resistance, shall not be terminated in the tensile zone unless one of the following conditions is satisfied:

a) The factored shear force at the point of termination does not exceed two-thirds of the factored shear resistance at that point.

b) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from the termination point equal to three-fourths the effective depth of the member. The excess stirrups shall be proportioned such that:

$$\frac{A_v f_y}{b_w s} \geq 0.40 \text{ MPa}$$

and the resulting stirrup spacing shall not exceed $\frac{d}{8\beta_b}$

c) Where size 35 or smaller bars are used, the continuing longitudinal steel provides double the area required for factored flexural resistance at the cutoff point and the factored shear force does not exceed three-fourths of the factored shear resistance.

8 — 10.1.2 **Positive Moment Reinforcement**

8 — 10.1.2.1 At least one-third of the positive moment reinforcement in simple members and one-fourth of the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. Such reinforcement shall extend at least 150 mm beyond the centreline of the support.

8 — 10.1.2.2 When a flexural member is part of the lateral load resisting system, the positive reinforcement required to be extended into the support by Clause 8 — 10.1.2.1 shall be anchored so as to develop the specified yield stress in tension, f_y, at the face of the support.

8 — 10.1.2.3 At simple supports and at points of inflection, postive moment tensile reinforcement shall be limited to a diameter such that ℓ_d computed from Clause 8 — 10.2 does not exceed:

$$\frac{M_r}{V} + \ell_a$$

The length ℓ_a at a support shall be the sum of the embedment length beyond the centre of the support and the equivalent embedment length of any furnished hook or mechanical anchorage. At a point of inflection, ℓ_a shall be limited to the effective depth of the member or 12 d_b, whichever is greater. The value M_1/V in the development length limitation may be increased by 30% when the ends of the reinforcing bars are confined by a compressive reaction.

8 — 10.1.3 **Negative Moment Reinforcement**

8 — 10.1.3.1 Tension reinforcement in a continuous, restrained, or cantilever member, or any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

8 — 10.1.3.2 At least one-third of the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, or $12d_b$, or 1/16 of the clear span, whichever is greatest.

8 — 10.1.4 **Special Members**

End anchorage shall be provided for tensile reinforcement in flexural members where the stress in the reinforcement is not directly proportional to moment at the ultimate limit state.

Development Length of Deformed Bars and Deformed Wire in Tension

8 — 10.2

8 — 10.2.1 The development length, ℓ_d, of deformed bars and deformed wire in shall be computed as the product of the basic development length tension given in Clause 8 — 10.2.2 and the applicable modification factor or factors given in Clauses 8 — 10.2.3 to 8 — 10.2.5, except that ℓ_d shall not be less than that specified in Clause 8 — 10.2.6.

8 — 10.2.2 The basic development length ℓ_d shall be:

a) For size 35 or smaller bars	$0.019\, A_b f_y/\sqrt{f'_c}$
but not less than:	$0.06\, d_b f_y$
b) For size 45 bars:	$26\, f_y/\sqrt{f'_c}$
c) For size 55 bars:	$34\, f_y/\sqrt{f'_c}$
d) For deformed wire:	$0.35\, d_b f_y/\sqrt{f'_c}$

8 — 10.2.3 The basic development length shall be multiplied by a factor of 1.40 for top reinforcement provided that at least 300 mm of concrete is cast below the top reinforcing bar.

8 — 10.2.4 When low-density concrete is used, the basic development lengths given in Clause 8 — 10.2.2, shall be multiplied by 1.33 for low-density concrete and 1.18 for semi low-density concrete, with linear interpolation when partial sand replacement is used, or the basic development length may be multiplied by $0.56\sqrt{f'_c}/f_{ct}$ but not less than 1.0, when f_{ct} is specified. The factors given in Clause 8 — 10.2.3 and 8 — 10.2.5 shall also be applied.

8 — 10.2.5 The basic development length may be multiplied by the following applicable factor or factors.

a) For reinforcement being developed in the length under consideration and spaced laterally at least 150 mm on centre and at least 75 mm from the side face of the member:	0.80
b) For reinforcement in flexural members in excess of that required:	(A_s required)/(A_s provided)
c) For reinforcement enclosed within a spiral made from bars not less than 6 mm in diameter with a pitch not more than 100 mm:	0.75

8 — 10.2.6 The development length, ℓ_d, shall be taken as not less than 300 mm except as specified in Clauses 8 — 10.9 and 8 — 10.10.3.

Development Length of Deformed Bars in Compression

8 — 10.3

8 — 10.3.1 The development length, ℓ_d, for bars in compression shall be computed from:

$$\ell_d = \frac{0.24\, f_y\, d_b}{\sqrt{f'_c}}$$

but shall not be less than $0.044\, f_y d_b$ nor less than 200 mm.

8 — 10.3.2 Where excess bar area is provided, the development length ℓ_d may be reduced by the ratio of area required to area provided. The development length may be reduced by 25% when the reinforcement is enclosed within spirals made from bars not less than 6 mm in diameter and with a pitch not more than 100 mm.

Development Length of Bundled Bars

8 — 10.4 The development length of each bar of bundled bars shall be that for the individual bar, increased by 20% for a three-bar bundle, and by 33% for a four-bar bundle.

Standard Hooks

8 — 10.5

8 — 10.5.1 Standard hooks shall be considered to develop a tensile stress in bar reinforcement of:

$$f_h = \xi \sqrt{f'_c}$$

where ξ shall not exceed the values given in Table 8 — 10.5.1. The value of ξ may be increased by 30% where enclosure is provided perpendicular to the plane of the hook. Enclosure may consist of external concrete or internal closed ties, spirals, or stirrups.

Table **ξ Values** **8 — 10.5.1**

Bar Size	f_y = 400 MPa		f_y = 300 MPa
	Top Bars	Other Bars	All Bars
10 to 15	45	45	30
20	37	45	30
25	30	45	30
30	30	40	30
35	30	35	30
45	27	27	27
55	18	18	18

8 — 10.5.2 An equivalent embedment length, ℓ_e, shall be computed using the provisions of Clause 8 — 10.2.2 by substituting f_h for f_y and ℓ_e for ℓ_d.

8 — 10.5.3 Hooks shall not be considered effective in adding to the compressive resistance of reinforcement.

Combination Development Length

8 — 10.6 Development length, ℓ_d, may consist of a combination of the equivalent embedment length of a hook or mechanical anchorage plus additional embedment length of the reinforcement measured from the point of tangency of the hook.

Development of Welded Wire Fabric in Tension

8 — 10.7

8 — 10.7.1 **Deformed Wire Fabric**

8 — 10.7.1.1 The development length, ℓ_d, of welded deformed wire fabric measured from the point of critical section to the end of the wire shall be computed as the product of the basic development length of Clauses 8 — 10.7.1.2 and 8 — 10.7.1.3 and the applicable modification factor or factors given in Clauses 8 — 10.2.3 to 8 — 10.2.5, but ℓ_d shall not be less than 200 mm except as provided for in Clause 8 — 10.10.4.

8 — 10.7.1.2 The basic development length of welded deformed wire fabric, with at least one cross-wire within the development length not less than 50 mm from the point of critical section, shall be:

$$\frac{0.36\, d_b\, (f_y - 138)}{\sqrt{f'_c}}$$

but not less than:

$$2.4 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f'_c}}$$

8 — 10.7.1.3 The basic development length of welded deformed wire fabric, with no cross-wires within the development length, shall be determined as for deformed wire.

8 — 10.7.2 **Smooth Wire Fabric**

The yield stress of welded smooth wire fabric shall be considered developed by embedment of two cross-wires with the closer cross-wire not less than 50 mm from the point of critical section. However, the development length, ℓ_d, measured from the point of critical section to the outermost cross-wire shall not be less than:

$$3.3 \frac{A_w}{s_w} \frac{f_y}{\sqrt{f'_c}}$$

modified by the ratio (A_s required)/(A_s provided) for reinforcement in excess of that required by analysis and by an appropriate factor given in Clause 8 — 10.2.4 for low-density concrete, but ℓ_d shall not be taken less than 150 mm except in the computation of lap splices in accordance with Clause 8 — 10.10.5.

Mechanical Anchorages

8 — 10.8 Any mechanical device capable of developing the strength of the reinforcement without damage to the concrete may be used as anchorage.

Anchorage of Shear Reinforcement

8 — 10.9

8 — 10.9.1 Shear reinforcement shall extend as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Shear reinforcement shall be anchored at both ends.

8 — 10.9.2 The ends of single leg, single U-stirrups, or multiple U-stirrups shall be anchored by one of the following means:

a) A standard hook plus an effective embedment of $0.50\ell_d$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, 0.50 d, and the start of the hook.

b) Embedment above or below the mid-depth, 0.50d, of the member on the compression side for a full development length, ℓ_d, but not less than 24 d_b or, for deformed bars or deformed wire, 300 mm.

c) For size 15 bar and D32 wire, and smaller, bending around the longitudinal reinforcement through at least 135°.

d) For each leg of welded plain wire fabric forming single U-stirrups, either:

   i) two longitudinal wires running at a 60 mm spacing along the beam at the top of the U; or

   ii) one longitudinal wire not more than 0.25 d from the compression face and a second wire closer to the compression face and spaced at least 50 mm from the first. The second wire may be beyond a bend or on a bend which has an inside diameter of at least 8 wire diameters.

8 — 10.9.3 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when the laps are $1.7\ell_d$. In members with depths of 450 mm or less, such splices with $A_b f_y$ not more than 40 000 N per leg may be considered adequate if the legs extend the full available depth of the member.

8 — 10.9.4 Between the anchored ends, each bend in the continuous portion of a transverse single U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.

8 — 10.9.5 Longitudinal bars bent to act as shear reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement, and in a compression zone shall be anchored above or below the mid-depth, 0.50 d, as specified for development length in Clause 8 — 10.2.

Splicing of Reinforcement

8 — 10.10 Splicing of reinforcement shall be made only as shown on the plans.

8 — 10.10.1 **Lap Splices**

8 — 10.10.1.1 Lap splices shall not be used for bars larger than size 35.

8 — 10.10.1.2 Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle, increased by 20% for a three-bar bundle and by 33% for a four-bar bundle. Individual bar splices within a bundle shall not overlap.

8 — 10.10.1.3 Bars spliced by non-contact lap splices in flexural members shall not be spaced transversely farther apart than 1/5 the required lap splice length, nor more than 150 mm.

8 — 10.10.2 **Welded Splices and Mechanical Connections**

8 — 10.10.2.1 Welded splices and other mechanical connections may be used. Except as provided in Clause 8 — 10.10.2, all welding shall conform to CSA Standard W186, *Welding of Reinforcing Bars in Reinforced Concrete Construction* and to the provisions of Section 11.

8 — 10.10.2.2 A full welded splice shall have bars butted and welded to develop in tension at least 125% of the specified yield stress, f_y, of the bar.

8 — 10.10.2.3 A full mechanical connection shall develop in tension or compression, as required, at least 125% of the specified yield stress, f_y, of the bar.

8 — 10.10.2.4 Welded splices and mechanical connections not meeting the requirements of Clauses 8 — 10.10.2.2 or 8 — 10.10.2.3, may be used in accordance with Clause 8 — 10.10.3.4.

8 — 10.10.3 **Splices of Deformed Bars and Deformed Wire in Tension**

8 — 10.10.3.1 The minimum length of lap for tension lap splices shall be as required for Class A, B, or C splices as given below but shall not be less than 300 mm

- Class A splice: $1.0\ell_d$
- Class B splice: $1.3\ell_d$
- Class C splice: $1.7\ell_d$

where ℓ_d is computed in accordance with Clause 8 — 10.2.

8 — 10.10.3.2 Lap splices of deformed bars and deformed wire in tension shall conform to the requirements of Table 8 — 10.10.3.2

Table **Tension Lap Splices** **8 — 10.10.3.2**

(A_s provided)/ (A_s required)	**Maximum Percent of A_s Spliced Within Required Lap Length**		
	50	**75**	**100**
equal to or greater than 2	Class A	Class A	Class B
less than 2	Class B	Class C	Class C

8 — 10.10.3.3 Welded splices or mechanical connections used in cases where the area of reinforcement provided is less than twice that required by analysis shall meet the requirements of Clause 8 — 10.10.2.2 or Clause 8 — 10.10.2.3.

8 — 10.10.3.4 Welded splices or mechanical connections used in cases where the area of reinforcement provided is at least twice that required by analysis shall satisfy the following requirements:

a) Splices shall be staggered at least 600 mm, and in such a manner as to develop at every section at least twice the calculated tensile force at that section, but not less than 140 MPa for the total area of reinforcement provided.

b) In computing tensile force developed at each section, spliced reinforcement may be rated at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to ℓ_d required to develop the specified yield strength f_y.

8 — 10.10.3.5 Splices in members subjected to axial tension shall be made with a full welded splice or full mechanical connection and be staggered at least $1.7\ \ell_d$.

8 — 10.10.4 **Splices of Deformed Bars in Compression**

8 — 10.10.4.1 Minimum length of lap for compression lap splices shall be $0.073\ f_y d_b$, but not less than 300 mm. For f'_c less than 20 MPa, the length of lap shall be increased by 33%.

8 — 10.10.4.2 In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than 0.0015 hs, the lap splice length may be multiplied by 0.83, but the lap length shall not be less than 300 mm. Tie legs perpendicular to the dimension h shall be used in determining the effective area.

8 — 10.10.4.3 In spirally-reinforced compression members, the lap splice length of bars within a spiral may be multiplied by 0.75, but the lap length shall not be less than 300 mm.

8 — 10.10.4.4 Welded splices or mechanical connections used in compression shall meet the requirements of Clause 8 — 10.10.2.2. or Clause 8 — 10.10.2.3.

8 — 10.10.5 **End Bearing Splices**

In bars required for compression only, compressive stress may be transmitted by bearing of square cut ends held in concentric contact by a suitable device. Bar ends shall terminate in flat surfaces within 1.5° of a right angle to the axis of the bars and shall be fitted within 3° of full bearing after assembly. End bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

8 — 10.10.6 **Splices of Welded Deformed Wire Fabric in Tension**

8 — 10.10.6.1 The minimum length of lap for lap splices of welded deformed wire fabric measured between the ends of each fabric sheet shall be the greater of $1.7\ell_d$ and 200 mm, and the overlap measured between outer crosswires of each fabric sheet shall not be less than 50 mm, where ℓ_d is the development length for the specified yield stress, f_y, in accordance with Clause 8—10.7.1.

8—10.10.6.2 Lap splices of welded deformed wire fabric, with no cross-wires within the lap splice length shall be determined as for deformed wire.

8—10.10.7 **Splices of Welded Smooth Wire Fabric in Tension**

8—10.10.7.1 The minimum length of lap for lap splices of welded smooth wire fabric shall be in accordance with Clauses 8—10.10.7.2 and 8—10.10.7.3.

8—10.10.7.2 When the area of reinforcement provided is less than twice that required by analysis at the splice location, the length of overlap measured between outermost cross-wires of each fabric sheet shall not be less than one spacing of cross-wires plus 50 mm, nor less than $1.5\ell_d$ nor less than 150 mm, where ℓ_d is the development length for the specified yield stress, f_y, in accordance with Clause 8—10.7.2.

8—10.10.7.3 When the area of reinforcement provided is at least twice that required by analysis at the splice location, the length of overlap measured between outermost cross-wires of each fabric sheet shall not be less than $1.5\ell_d$ and not less than 50 mm, where ℓ_d is the development length for the specified yield stress, f_y, in accordance with Clause 8—10.7.2.

Composite Construction — 8—11

General — 8—11.1

8—11.1.1 The design of composite concrete members shall conform to the provisions of other clauses in Section 8 except as modified in Clause 8—11.

8—11.1.2 The total depth of the composite member shall be used in computing the shear and flexural resistances.

8—11.1.3 If the properties of the various components are different, the most critical values shall be used.

8—11.1.4 A member shall not be regarded as composite until the concrete components have attained a strength of $0.75\ f'_c$.

8—11.1.5. For shored members, shoring shall not be removed until the supported components have developed the specified strength to support the loads and to limit deflections and cracking at the time of shoring removal.

Shear — 8—11.2

8—11.2.1 The factored shear resistance of a composite section shall be computed in accordance with Clause 8—8.2 as for a monolithically-cast member of the the same cross-sectional shape.

8—11.2.2 The proportioning for factored interface shear shall be in accordance with Clause 8—8.7.

T-Girders — 8—12

General — 8—12.1

In T-girder construction, the girder web and slab shall be built integrally or otherwise effectively bonded together. The interface shear at the junction of the flange and the web shall be provided for in accordance with Clause 8—8.7. Where applicable, the requirements of Clause 8—11 for composite concrete members shall apply.

Effective Flange Width — 8—12.2

The effective flange width to be used in design shall be as specified in Section 3.

Isolated Girders — 8—12.3

Isolated girders, in which the T-form is used to provide additional compression area, shall have a flange thickness not less than one-half the width of the girder web and a total flange width not more than four times the width of the girder web.

Flange Thickness — 8—12.4

The thickness of the top flange shall be sufficient to resist flexure in the transverse direction and shall not be less than 1/16 of the clear distance between the webs nor less than 225 mm.

Diaphragms — 8—12.5

8—12.5.1 Diaphragms shall be provided at abutments and piers. The diaphragms shall be proportioned to transfer loads to the supports and to allow for future jacking of girders.

8—12.5.2 Intermediate diaphragms shall be provided if required for purposes of load distribution in accordance with the requirements of Section 3.

Box Girders — 8—13

General — 8—13.1

In box-girder construction, the girder web and top and bottom flanges shall be build integrally or otherwise effectively bonded together. Full transfer of shear forces shall be assured at the interface of the girder webs with the top and bottom flanges. The proportioning for interface shear shall be in accordance with Clause 8—8.7.

Changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness.

Effective Flange Width **8—13.2** The effective flange width to be used in design shall be as specified in Section 3.

Flange Thickness **8—13.3**

8—13.3.1 The thickness of the top flange shall be sufficient to resist flexure and shall not be less than 1/16 of the clear distance between the webs nor less than 225 mm.

8—13.3.2 The thickness of the bottom flange shall not be less than 1/16 of the clear distance between girder webs nor less than 150 mm.

8—13.3.3 **Top and Bottom Flange Reinforcement**

8—13.3.3.1 A minimum reinforcement of 0.40% of the flange area shall be placed in the bottom flange parallel to the girder span. The maximum spacing of the reinforcement shall not exceed 450 mm.

8—13.3.3.2 A minimum reinforcement of 0.60% of the cross-sectional area of the flange based on the least flange thickness, shall be placed in both top and bottom flanges transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 300 mm for the top slab and 450 mm for the bottom slab. All transverse reinforcement in the bottom flange shall extend to the exterior face of the outside girder web and be anchored by a standard 90° hook.

8—13.3.3.3 At least one-third of the bottom layer of the transverse reinforcement in the top flange shall extend to the exterior face of the outside girder web and be anchored by a standard 90° hook. If the flange extends beyond the last girder web, such reinforcement shall extend into the flange overhang and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook.

Diaphragms **8—13.4**

8—13.4.1 Diaphragms shall be provided at abutments and piers. The diaphragms shall be proportioned to transfer loads to the supports and to allow for future jacking of girders.

8—13.4.2. Intermediate diaphragms shall be provided if required for purposes of load distribution in accordance with the requirements of Section 3.

Concrete Arches **8—14**

Shape of Arch Rings **8—14.1** The shape of arch rings shall be selected so as to minimize bending under the effect of combined dead and live loads.

Spandrel Walls **8—14.2** When the spandrel walls of filled spandrel arches exceed 2.50 m in height above the extrados they shall be supported by transverse diaphragm walls or deep counterforts having a back slope not less than 45° with the vertical.

Expansion Joints **8—14.3** Vertical expansion joints shall be placed in the spandrel walls of arches to provide for movement due to temperature change and arch deflection. Such joints shall be placed at the ends of spans and at intermediate points, preferably not more than 15 m apart.

Reinforcement **8—14.4** Arch ribs shall be reinforced with a complete double line of longitudinal reinforcement consisting of an intradosal system and extradosal system connected by a series of stirrups and ties.

For barrel arches, a system of transverse reinforcement fully-anchored to the longitudinal reinforcement shall be used in intrados and extrados. The transverse reinforcement shall be proportioned to resist flexure and axial tension due to overturning action of the spandrel wall. For rib arches, the longitudinal rib reinforcement shall be enclosed by lateral hoops or ties as for compression members.

Drainage of Spandrel Fill **8—14.5** The fills of filled spandrel arches shall be effectively drained.

Special Detailing Provisions **8—15**

Footings **8—15.1**

8—15.1.1 General

Footings shall be proportioned to meet the requirements of Sections 6 and 8 except as modified in Clause 8 — 15.1.

8 — 15.1.2 **Flexure**

8 — 15.1.2.1 The moment at any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and by computing the moment of the forces acting over the entire area of the footing on one side of said plane.

8 — 15.1.2.2 The greatest bending moment to be used in proportioning an isolated footing shall be the moment computed at sections located as follows:

a) for footings supporting a concrete column, pedestal or wall, the face of the column, pedestal or wall,

b) for footings under steel bases, halfway between the face of the column or pedestal and the edge of the steel base.

8 — 15.1.2.3 In one-way reinforced footings and in two-way square reinforced footings, the reinforcement shall be distributed uniformly across the full width of the footing.

8 — 15.1.2.4 In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. The reinforcement in the short direction shall be uniformly distributed across a band width, b, centred with respect to the centreline of the column or pedestal and having a width equal to the length of the short side of the footing. The reinforcement in the band width, b, shall be determined as follows:

$$\frac{\text{Reinforcement in Band Width, } b}{\text{Total Reinforcement in Short Direction}} = \frac{2}{(\beta + 1)}$$

8 — 15.1.3 **Shear**

8 — 15.1.3.1 The proportioning of footings for shear shall be in accordance with Clause 8 — 8.8.

8 — 15.1.3.2 In computing shear force at any section of a footing supported on piles, the entire reaction from any pile whose centre is located a distance of 0.5 d_p or more outside the section shall be assumed to produce shear on the section.

Piers, Abutments and Retaining Walls **8 — 15.2**

8 — 15.2.1 Contraction joints shall be provided in accordance with the requirements of Section 6.

8 — 15.2.2 Concrete pedestals for bearing seats shall be anchored to the abutments and pier caps by reinforcement.

Mass Concrete **8 — 15.3** Mass concrete components shall be detailed to control cracking due to shrinkage and temperature.

Structures Subject to Earthquake Forces **8 — 15.4** Structures subject to earthquake forces shall be detailed in accordance with the special provisions for seismic design in CSA Standard CAN 3-A23.3, *Code for the Design of Concrete Structures for Buildings*.

Concrete Piles **8 — 16**

General **8 — 16.1** The design of concrete piles shall conform to the requirements of Sections 6 and 8 except as modified in Clause 8 — 16.

Concrete piles may be precast or cast-in-place members.

Specified Concrete Strength **8 — 16.2** The minimum specified concrete strength shall be 35 MPa.

Handling **8 — 16.3** Precast piles shall be investigated for conditions during handling and transportation, taking into account the weight of the pile plus a 50% allowance for impact.

Splices **8 — 16.4**

8 — 16.4.1 The strength of the splices shall be at least equal to the strength of the pile in compression, tension, or flexure.

8 — 16.4.2 The shape and size of splices shall be such as not to affect the performance of the piles.

8 — 16.4.3 The slack in mechanical splices shall be less than 0.50 mm in either compression or tension.

Pile Dimensions **8 — 16.5** The minimum diameter or side dimension for precast piles shall be 200 mm, and for cast-in-place piles shall be 400 mm.

Reinforcement Details **8 — 16.6**

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8 — 16.6.1 For precast concrete piles, the area of longitudinal reinforcement shall not be less than 1.50% nor more than 8% of the cross-sectional area of the pile. At least six longitudinal bars shall be used for round or octagonal piles and at least four bars for square piles.

For piles up to 600 mm in diameter, the spiral wire shall be at least 5 mm diameter. Spiral reinforcement at the ends of these piles shall have a pitch of 75 mm for 16 turns. In addition, the top 150 mm of pile shall have five turns of spiral winding at 25 mm pitch. For the remainder of the pile, the longitudinal reinforcement shall be enclosed with spiral reinforcement with not more than 150 mm pitch.

For piles having a diameter greater than 600 mm, the spiral wire shall be at least 6 mm diameter. Spiral reinforcement at the ends of these piles shall have a pitch of 50 mm for 16 turns. In addition, the top 150 mm shall have four turns of spiral winding at 40 mm pitch. For the remainder of the pile, the longitudinal reinforcement shall be enclosed with spiral reinforcement with not more than 100 mm pitch.

The spiral wires shall be in accordance with CSA Standard G30.3, *Cold-Drawn Steel Wire for Concrete Reinforcement*.

8 — 16.6.2 For cast-in-place piles, the longitudinal reinforcement shall consist of at least four bars.

8 — 16.6.3 **Concrete Protection for Reinforcement**

The minimum cover for pile reinforcement shall be in accordance with Clause 8 — 9.3.

Concrete Poles

8 — 17

General

8 — 17.1 The design of reinforced concrete poles shall conform to the requirements of Sections 5 and 8 except as modified in Clause 8 — 17.

The poles shall be monolithic, whether cast or spun, and shall taper uniformly from bottom to top.

Specified Concrete Strength

8 — 17.2 The minimum specified concrete strength shall be 35 MPa.

Handling

8 — 17.3 Poles shall be investigated for conditions during handling, transportation and erection taking into account the weight of the pole plus 50% allowance for impact.

Factored Resistance

8 — 17.4

8 — 17.4.1 The resistance of poles subjected to combined flexure and torsion shall be such that:

$$\frac{M}{M_u} + \left(\frac{T}{T_u}\right)^2 \leq 1.0$$

Deflection

8 — 17.5 The long-term lateral deflection at the top of the pole under dead load shall not exceed 1% of the pole height above the ground.

Reinforcement Details

8 — 17.6

8 — 17.6.1 The reinforcement shall be assembled in cages securely fastened so that the steel will be accurately held in its proper position.

8 — 17.6.2 Longitudinal bars shall, where possible, be continuous throughout their length and in any event shall not be lapped more than once. The length of the lap shall not be less than the development length, ℓ_d, specified in Clause 8 — 10.2. Laps shall be staggered a minimum of 40 times the diameter of the smaller bar, except that diametrically opposite bar lines may be lapped at the same locations.

8 — 17.6.3 Two layers of reinforcing wire shall be wound helically around the longitudinal reinforcement for the full length of the pole. One layer shall be wound clockwise and the other counterclockwise.

The diameter of the wire shall not be less than 3 mm. The pitch of each helix shall not be greater than 300 mm. The pitch for a distance equal to two times the pole diameter from the top shall not exceed one-half the pole diameter. The spiral reinforcement shall meet the requirements of CSA Standard G30.3, *Cold-Drawn Steel Wire for Concrete Reinforcement*, and shall be galvanized to provide a zinc coating of at least 5% by pickup weight.

8 — 17.6.4 **Concrete Protection for Reinforcement**

The minimum cover for pole reinforcement shall be in accordance with Clause 8 — 9.3 unless otherwise approved.

8 — 17.6.5 Any chairs or other devices used between the steel and the forms shall be made of non-corrosive material and shall not deface or weaken the poles.

9 Prestressed Concrete

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Reference Publications

Section 9 makes reference to the following publications:

CSA Standards

CAN3-A23.1-M77	*Concrete Materials and Methods of Concrete Construction*
CAN3-A23.2-M77	*Methods of Test for Concrete*
G279-M1982	*Steel for Prestressed Concrete Tendons*
A251-M1982	*Qualification Code for Manufacturers of Architectural and Structural Precast Concrete*
G30.3-1972(R1979)	*Cold-Drawn Steel Wire for Concrete Reinforcement*

Scope 9—1

General 9—1.1

Section 9 provides requirements for the design of prestressed concrete members. Unless otherwise modified by the provisions of this section, the design shall also conform to the requirements of Section 8.

Types of Structures 9—1.2

Provisons in this section apply to bridge components, retaining walls, piles and poles made of normal-density or low-density concrete and prestressed with high-strength steel which is bonded to the concrete.

Definitions 9—2

Anchorage — a device by which the prestressing force is transferred to the concrete.

At Jacking — at the time of tensioning the tendons.

At Transfer — immediately after transfer.

Bonded Tendon — a tendon that is bonded to the concrete either directly or by means of grouting.

Camber — a deflection of a prestressed concrete member due to prestressing force and dead loads.

Concrete Cover — the least distance between the surface of reinforcing bars, strands, or post-tensioning ducts and the surface of concrete.

Creep — a time-dependent deformation of concrete under sustained load.

Curvature Friction — the friction resulting from the curvature of the specified profile of post-tensioning tendons.

Development Length — the length of embedment required to develop the specified strength of reinforcement at a critical section.

Effective Prestress — the stress or force remaining in the tendons after all losses have occurred.

Jacking Force — the force exerted by the device which introduces tension into the tendons. For pretensioning, the specified jacking force excludes the force to compensate for the slip of the anchorage and temperature correction. For post-tensioning, the specified jacking force includes the force to compensate for anchorage slip.

Multi-Beam Decks — deck systems consisting of precast members placed side by side and overlaid by a cast-in-place structural slab to provide shear transfer between adjacent units.

Partially Prestressed Concrete — prestressed concrete which is assumed to crack in flexure under serviceability limit states Type II loads.

Post-Tensioning — prestressed concrete in which the tendons are tensioned after the concrete has hardened.

Precast Members — concrete members produced in plants certified in accordance with CSA Standard A251, *Qualification Code for Manufacturers of Architectural and Structural Precast Concrete*.

Prestressed Concrete — reinforced concrete in which internal stresses and deformations are intially introduced, of such magnitude and distribution that the subsequent stresses and deformations resulting from dead and live loads are counteracted to a desired degree.

Pretensioning — prestressed concrete in which the tendons are tensioned before the concrete is placed.

Primary Prestressing Effects — the effects, other than secondary prestressing effects, due to the prestressing force.

Primary Prestressing Moment — the product of the prestressing force and its eccentricity.

Relaxation — the time-dependent reduction of stress in steel at constant strain.

Secondary Prestressing Effects — the effects caused by restraint of deformation due to the prestressing force.

Segmental Girder — a girder made up of individual components post-tensioned together to act as a monolithic unit under loads.

Steel Stress Range — the algebraic difference between the maximum and minimum stresses for non-prestressed reinforcement or the increase in tension for prestressed reinforcement, due to movement of a design truck, at serviceability limit states Type I.

Tendon — a high-strength steel element used to impart prestress to the concrete.

Transfer — the operation of imparting the force in the tendons to the concrete.

Transfer Length — the length over which the prestressing force is transferred to the concrete by bond in a pretensioned member.

Wobble Friction — the friction caused by the unintentional deviation of a post-tensioning tendon from its specified profile.

Notation 9—3

ANC	the loss of prestress due to slip of prestressing steel at anchorage, MPa
A_{bs}	the area of reinforcement located perpendicular to the plane of potential bursting crack, mm^2
A_g	the gross cross-sectional area, mm^2
A_ℓ	the area of longitudinal reinforcement, mm^2
A_o	the area enclosed by lines connecting the centres of the longitudinal reinforcing bars inside the closed stirrups or spirals, mm^2
A_{ps}	the area of prestressed reinforcement in tension zone, mm^2
A_s	the area of non-prestressed tension reinforcement, mm^2
A'_s	the area of non-prestressed compression reinforcement, mm^2
A_{sf}	the area of steel required to develop the compressive resistance of the overhanging portions of the flange of a flanged section, mm^2
A_{sr}	the area of steel required to develop the compressive resistance of the web of a flanged section, mm^2
A_t	the area of one leg of a closed stirrup or spiral resisting torsion within a distance s, mm^2
A_v	the area of shear reinforcement within a distance s, mm^2
a_1	the side dimension of a square post-tensioning anchor, or lesser dimension of a rectangular post-tensioning anchor, or side dimension of a square equivalent in area to a circular post-tensioning anchor, mm
a_2	the minimum distance between the centrelines of anchors, or twice the distance from the centreline of the anchor to the nearest edge of concrete, whichever is less, mm
b	the width of the compression face of a member, mm
b_b	the width of a concrete section in the plane of a potential bursting crack, mm
b_e	the equivalent width at the level of the ducts of a web with post-tensioning ducts, mm
b_o	the periphery of the critical section for slabs and footings for shear, mm
b_w	the width of web or width of cross-section being investigated for shear, mm
CR	the loss of prestress due to creep of concrete, MPa
C_m	a factor relating the actual moment diagram to an equivalent uniform moment diagram
c	the distance from the centroidal axis of a pile to the extreme fibre either in tension or in compression, mm
d	the distance from the extreme compressive fibre to the centroid of the prestressed and non-prestressed reinforcement, mm
d_b	the nominal diameter of a prestressing strand, mm
d_d	the outside diameter of a post-tensioning duct, mm
E	a modulus of elasticity, MPa
E_c	the modulus of elasticity of concrete calculated in accordance with Clause 8—5.4.1, MPa
E_{ci}	the modulus of elasticity of concrete at transfer calculated in accordance with Clause 8—5.4.1 using f'_{ci}, MPa
EI	the flexural stiffness of compression members, $N\cdot mm^2$
ES	the loss of prestress due to elastic shortening of concrete, MPa
E_s	the modulus of elasticity of prestressed reinforcement, MPa
e	the base of Napierian logarithms; or the eccentricity of prestressed reinforcement, mm
FR	the loss of prestress due to friction at a point x metres from the jacking end, MPa
F_{bs}	the total bursting force behind a post-tensioning anchor, N
F_{sj}	the jacking force in a post-tensioning tendon, N
f_{bs}	the bursting stress behind a post-tensioning anchor, MPa
f'_c	the specified compressive strength of concrete, MPa
f'_{ci}	the compressive strength of concrete at transfer, MPa
f_{cds}	the concrete stress at the centre of gravity of prestressing steel due to all dead loads except the dead load present at transfer at the same section or sections for which f_{cir} is calculated, MPa. Values of f_{cds} are positive when they are tensile stresses.
f_{cir}	the concrete stress at the centre of gravity of prestressing steel, due to the prestressing force at transfer and the self-weight of the member at sections of maximum moment, MPa.
f_d	the stress due to dead load at the extreme fibre of a section at which tensile stresses can be caused by live load, MPa.

$f_{\ell y}$	the yield stress of longitudinal reinforcement; for prestressing steel this is equal to f_{py}, MPa.
f_{pc}	the compressive stress in concrete after all prestress losses have occurred, either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange, MPa. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.
f_{pe}	the compressive stress in concrete due to effective prestress, including secondary effects, at the extreme fibre of a section at which tensile stresses can be caused by live load, MPa.
f_{ps}	the calculated stress in prestressed reinforcement at the ultimate limit state, MPa.
f_{pu}	the specified strength of prestressing steel, MPa.
f_{py}	the yield stress of prestressing steel; this may be taken as 0.85 f_{pu} for stress-relieved strands and wires, and 0.90 f_{pu} for low-relaxation strands and wires, MPa.
f_r	the flexural tensile stress at cracking, calculated in accordance with Clause 8—5.5, MPa.
f_{ri}	the flexural tensile stress at cracking, calculated in accordance with Clause 8—5.5 using f'_{ci}, MPa.
f_{se}	the effective stress in the prestressing steel after losses, MPa.
f_{si}	the stress in the prestressing steel immediately before transfer, MPa.
f_{sj}	the stress in the prestressing steel at jacking, MPa.
f_{st}	the stress in the prestressing steel at transfer, MPa.
f_y	the specified yield stress of non-prestressed reinforcement, MPa.
H	the mean annual relative humidity, percent.
h	the overall thickness of a member, mm.
h_f	the average thickness of the flange of a flanged member, mm.
I	the moment of inertia of the section resisting live loads, mm^4.
I_g	the moment of inertia of the gross concrete section about the centroidal axis, mm^4.
K	a wobble friction coefficient per metre of prestressing tendon.
K_{cr}	a factor used in Clause 9—9.2.3.2 to compute prestress loss due to creep of concrete.
K_1	a constant used to evaluate the flexural resistance of a pile.
k	an effective length factor for compression members.
L_b	the flexural bond length of a prestressing strand, mm.
L_d	the development length of a prestressing strand, mm.
L_t	the transfer length of a prestressing strand, mm.
ℓ	length, mm.
ℓ_e	effective length, mm.
ℓ_u	the unsupported length of a compression member, mm.
M	the factored flexural moment at a section at the ultimate limit state, N·mm.
M_a	the allowable flexural moment on a pile without axial load, at a serviceability limit state, N·mm.
M_c	the magnified moment used for proportioning slender compression members, N·mm.
M_{cr}	the moment causing flexural cracking at the section under consideration, N·mm.
M_{max}	the maximum bending moment due to live load at the section under consideration, N·mm.
M_{ser}	the flexural moment at a pile section at a serviceability limit state, N·mm.
M_u	the factored flexural resistance of the section under consideration, N·mm.
M_1	the value of the smaller end moment at ultimate limit state due to factored load acting on a compression member, positive if the member is bent in single curvature and negative if bent in double curvature N·mm.
M_2	the value of the larger end moment at ultimate limit state due to factored load acting on a compression member, always positive, N·mm.
N	the total number of post-tensioning tendons.
P	the factored axial load at a section at the ultimate limit state, N.
P_a	the allowable axial load on a compression member without flexure, at a serviceability limit state, N.
P_c	the critical load defined in Clause 9—14.2.2(c), N.

P_o	the factored axial resistance of a section in pure compression, N.
P_{ser}	the axial load at a pile section at a serviceability limit state, N.
P_u	the factored resistance of a section in compression, N.
p_o	the perimeter of the area A_o, mm.
R	a reduction factor for laterally-unsupported piles defined in Clause 9—27.6.
REL_1	the loss of prestress due to relaxation of prestressing steel prior to transfer, MPa.
REL_2	the loss of prestress due to relaxation of prestressing steel after transfer, MPa.
r	the radius of gyration of gross cross-section, mm.
SH	the loss of prestress due to shrinkage of concrete, MPa.
s	the spacing of stirrups, mm; or the pitch of spirals, mm.
T	the torsional moment at a section at the ultimate limit state, N·mm.
T_u	the factored torsional resistance of a section under consideration N·mm.
t	time, days.
V	the shear force at a section at the ultimate limit state, N.
V_c	the shear resistance provided by concrete, N.
V_{ci}	the shear resistance at flexural-shear cracking due to all loads, when such cracking results from combined shear and moment, N.
V_{cw}	the shear resistance at web cracking due to all loads, when such cracking results from principal tensile stresses in the web, N.
V_d	the shear force due to unfactored dead load at a section, N.
V_l	the shear force due to live load occurring simultaneously with M_{max} at a section, N.
V_p	the shear component of the effective prestress force at a section under consideration, N.
V_s	the shear resistance provided by shear reinforcement, N.
V_u	the factored shear resistance of a section, N.
X_m	the location of maximum bursting stress, measured from the loaded face of the end block, mm.
x	the length of a prestressing tendon from the jacking end to any point under consideration, m.
y_t	the distance from the centroidal axis of the gross section resisting live load, to the extreme fibre in tension, mm.
α	the vector sum of angular changes in elevation and plan of a prestressing tendon profile from the jacking end to any point x, rad.
β_d	the ratio of maximum factored dead load moment to maximum factored total load moment, always positive.
γ	a non-dimensional section parameter defined in Clause 9—20.2.
Δf_s	the total loss of prestress, MPa.
Δf_{s1}	the loss of prestress at transfer, MPa.
Δf_{s2}	the loss of prestress after transfer, MPa.
δ	a moment magnification factor for compression members defined in Clause 9—14.2.2.
δ_i	the camber at transfer, mm.
δ_t	the camber at any time, t, days after transfer, mm.
δ_u	the long-term camber, mm.
λ	a constant used to compute the flexural stiffness of compression members, EI, defined in Clause 9—14.2.2.
μ	a coefficient of friction.
ρ	the ratio of non-prestressed tension reinforcement, A_s/bd.
ρ_{bs}	the ratio of non-prestressed reinforcement located perpendicular to the plane of potential bursting crack, $A_{bs}/b_b s$.
ρ_p	the ratio of prestressed reinforcement, A_{ps}/bd.
ϕ	a performance factor.
ψ	a parameter used in Clause 9—17.1.2 to evaluate end block stresses.

Limit States 9—4

General 9—4.1

Structural components shall be proportioned to satisfy the requirements at serviceability limit states and the ultimate limit states.

Serviceability Limit States 9—4.2

9—4.2.1 The serviceability limit states Type I to be considered shall be those of vibration and fatigue.

Structural components shall be proportioned to meet the vibration requirements as specified in Section 2, and for partial prestressing, the stress range requirements specified in Clause 9—8.3.

9—4.2.2 The serviceability limit states Type II to be considered shall be those of concrete stresses and deformations.

Except as permitted in Clause 9—8.3, the tensile and compressive stresses in the concrete, calculated on the basis of uncracked section, shall not exceed the values specified in Clauses 9—8.1, 9—8.2, 9—27.6.2 and 9—28.4.2.

Short-term and long-term deformations which may adversely affect the function of the structure shall be considered.

Ultimate Limit States

9—4.3 The ultimate limit state to be considered shall be that of strength. Structural components shall be proportioned so that the factored resistances are greater than or equal to the load effect of factored loads specified in Section 2.

General Considerations

9—5

Design

9—5.1 Components shall be proportioned for all load stages that may be critical during the life of the structure, including construction.

The concrete stresses shall not exceed the values specified in Clause 9—8.

Stress Concentrations

9—5.2 Stress concentrations due to prestressing or other loads or restraints shall be considered.

Buckling

9—5.3 Buckling of a member prior to grouting, buckling during handling and erection and buckling of thin webs and flanges shall be considered.

Minimum Prestress

9—5.4 Sections with an average effective prestress of less than 1.50 MPa shall be proportioned in accordance with the requirements of Section 8.

Basic Assumptions

9—6

At Serviceability Limit States

9—6.1 The following assumptions shall be used in investigating sections after transfer of prestress and at loads prior to cracking:

a) Strains are directly proportional to distance from the neutral axis.

b) Stress is directly proportional to strain.

c) In calculating section properties prior to grouting, the loss of concrete area due to post-tensioning ducts, coupler sheaths or transition trumpets may be neglected, except where such loss of area is significant.

The transformed area of bonded reinforcement may be included in pretensioned members and in post-tensioned members after grouting.

At Ultimate Limit States

9—6.2 At ultimate limit states the assumptions given in Clause 8—7.2 shall apply, except that Clause 8—7.2.4 shall apply only to non-prestressed reinforcement conforming to Clause 8—4.2.

Specified Strengths

9—7

Concrete

9—7.1

9—7.1.1 **Specified Strength**

Design shall be based on specified strengths of concrete of 30 to 40 MPa. Higher strengths, up to 55 MPa, may be used if approved.

9—7.1.2 **Strength at Transfer**

Concrete shall not be prestressed until the compressive strength, determined in accordance with CSA Standard CAN3-A23.1, *Concrete Materials and Methods of Concrete Construction*, is at least 25 MPa for pretensioned members and 20 MPa for post-tensioned members.

Grout

9—7.2

9—7.2.1 The specified compressive strength of grout for post-tensioning ducts shall be 35 MPa and shall be based on 50 mm cubes made and tested in accordance with CSA Standard CAN3-A23.2, *Methods of Test for Concrete*, at 28 days; grout of an equivalent strength established by other approved means may be used.

9—7.2.2 After grouting of post-tensioning ducts, load shall not be applied to or removed from the components until the grout has reached a compressive strength of at least 20 MPa.

Prestressing Steel

9—7.3

9—7.3.1 Prestressing steel shall be in the form of high tensile strength wire, high tensile strength strand, or high strength bar and shall meet the requirements of CSA Standard G279, *Steel for Prestressed Concrete Tendons*.

9 — 7.3.2 The modulus of elasticity of prestressing steel shall be based on representative stress-strain curves furnished by the manufacturer. In the absence of such data, the following values may be used for both stress-relieved and low-relaxation steels.

- a) High tensile strength wire: 200 000 MPa
- b) Seven-wire high-strength strand,
  - Size Designation 13: 200 000 MPa
  - Size Designations 15 and 16: 195 000 MPa
- c) High-strength bar: 205 000 MPa

Post-Tensioning Anchorages and Couplers 9 — 7.4

9 — 7.4.1 **Anchorages**

Anchorages, when tested in an unbonded condition, shall develop at least 95% of the ultimate strength of the tendons without exceeding the anticipated set. After tensioning and seating, anchorages shall sustain applied loads without slippage, distortion or other changes which result in loss of prestress. The dimensions of the anchorages shall be established by the manufacturer of the post-tensioning system. Details of anchorages shall be based on the maximum applied jacking force and the specified strength of concrete at transfer.

9 — 7.4.2 **Couplers**

Couplers, when tested in an unbonded condition, shall develop 100% of the ultimate strength of the tendons without exceeding the anticipated set.

Concrete Stress Limitations 9 — 8

At Transfer 9 — 8.1

Stresses at transfer shall not exceed the following:

- a) Compression: $0.60f'_{ci}$
- b) Tension in members without bonded non-prestressed reinforcement in the tension zone: $0.40f_n$
- c) Tension in members with bonded non-prestressed reinforcement in the tension zone: $1.0f_n$

  Where the calculated tensile stress is between $0.40f_n$ and $1.0f_n$, reinforcement shall be provided to resist the total tensile force in the concrete computed on the basis of an uncracked section. The stress in the reinforcement shall not exceed $0.60f_y$ or 200 MPa, whichever is smaller.
- d) Tension at joints in segmental members:
  - i) Without bonded non-prestressed reinforcement passing through the joint in the tension zone: zero
  - ii) With bonded non-prestressed reinforcement passing though the joint in the tension zone: $0.40f_n$

  Where the calculated tensile stress is between zero and $0.40f_n$, reinforcement shall be provided to resist the total tensile force in the concrete computed on the basis of an uncracked section. The stress in the reinforcement shall not exceed $0.60f_y$ or 200 MPa, whichever is smaller.

Under Loads 9 — 8.2

Stresses at serviceability limit states Type II loads, after allowance for all losses and redistribution of load effects, shall not exceed the following:

- a) Compression:
  - i) Precast Concrete Members: $0.45f'_c$
  - ii) Cast-in-Place Members: $0.40f'_c$
- b) Average compressive stress due to prestress alone, in post-tensioned decks with round voids: $0.16f'_c$
- c) Tension
  - i) In precompressed tensile zones subject to severe exposure conditions, such as coastal areas or tops of unprotected bridge decks, or members in axial tension: zero
  - ii) In precompressed tensile zones of segmental members without bonded non-prestressed reinforcement passing through the joints: zero
  - iii) In precompressed tensile zones subject to moderate exposure conditions, such as top of bridge decks with waterproofing and asphalt: $0.40f_r$
  - iv) Other precompressed tensile zones: $0.80f_r$
- d) Tension in other areas shall be limited by the allowable stresses at transfer in

accordance with Clause 9—8.1(b) to 9—8.1(d).

Partial Prestressing **9—8.3**

The tensile stress limits specified in Clause 9—8.2(c) may be waived if calculations based on experimentally verified rational procedures demonstrate that:

a) the steel stress range does not exceed 125 MPa; and

b) the durability of the resulting partially prestressed structure is at least comparable to that of an equivalent non-prestressed structure designed in accordance with the requirements of Section 8.

Prestressing Requirements 9—9

Stress Limitations **9—9.1**

The stress in prestressing steel shall not exceed the values shown in Table 9—9.1.

The maximum stress at jacking shall in no case exceed $0.94f_{py}$.

Table **Prestressing Steel Stress Limitations** **9—9.1**

	Type of Steel		
	Stress-Relieved Strand and Wire	**Low-Relaxation Strand and Wire**	**High-Strength Bar**
a) At Jacking			
Pretensioning	$0.80f_{pu}$	$0.80f_{pu}$	—
Post-Tensioning	$0.80f_{pu}$	$0.85f_{pu}$	$0.80f_{pu}$
b) At Transfer			
Pretensioning	$0.70f_{pu}$	$0.75f_{pu}$	—
Post-Tensioning	$0.70f_{pu}$	$0.75f_{pu}$	$0.70f_{pu}$

Loss of Prestress **9—9.2**

9—9.2.1 **General**

In computing the prestressing forces in the tendons, the loss of prestress due to the following shall be considered:

a) anchorage slip and friction,

b) relaxation of prestressing steel,

c) elastic shortening of concrete,

d) creep of concrete,

e) shrinkage of concrete, and

f) any other special circumstances.

For multi-stage construction and/or multi-stage prestressing, the prestress losses shall be computed considering the elapsed time between each stage. In lieu of a more detailed analysis, for members constructed and prestressed in a single stage, the prestress losses at transfer, Δf_{s1}, and after transfer, Δf_{s2}, shall be considered and calculated in accordance with Clauses 9—9.2.2, and 9—9.2.3. The total loss considered shall be:

$$\Delta f_s = \Delta f_{s1} + \Delta f_{s2}$$

9—9.2.2 **Losses at Transfer**

9—9.2.2.1 The losses at transfer shall be taken as:

$$\Delta f_{s1} = ANC + FR + REL_1 + ES$$

9—9.2.2.2 **Anchorage Slip**

The magnitude of the anchorage slip shall be either as required to control the stress in the prestressing steel at transfer, or as recommended by the manufacturer of the anchorage, whichever is greater. The magnitude of the slip shall be shown on the plans.

9—9.2.2.3 **Friction Loss**

Loss due to friction, FR, between prestressing steel and the duct enclosure at a distance x from the jacking end shall be calculated from:

$$FR = f_{sj}[1 - e^{-(Kx + \mu\alpha)}]$$

Values of K and μ shall be based on experimental data for the materials specified and shall be shown on the plans. In the absence of such data, the value of K and μ given in Table 9—9.2.2.3 may be used.

9—9.2.2.4 **Relaxation of Prestressing Steel**

In pretensioned members the relaxation loss in prestressing steel initially stressed in excess of $0.50f_{pu}$ shall be calculated as follows.

a) Stress-Relieved Strand and Wire:

$$REL_1 = \frac{\log(24t)}{10}\left[\frac{f_{si}}{f_{py}} - 0.55\right] f_{si}$$

b) Low-Relaxation Strand and Wire:

$$REL_1 = \frac{\log(24t)}{45}\left[\frac{f_{si}}{f_{py}} - 0.55\right] f_{si}$$

Table **Friction Factors** **9 — 9.2.2.3**

Steel Sheath Type	Strand		Wire		Smooth Bar		Threaded Bar	
	K	μ	K	μ	K	μ	K	μ
Rigid	0.0016	0.18	0.0010	0.16	—	—	—	—
Semi-Rigid over 75 mm O.D.	0.0033	0.20	0.0016	0.18	—	—	—	—
Semi-Rigid up to 75 mm O.D.	0.0046	0.20	0.0023	0.18	0.0010	0.18	0.0026	0.30

9 — 9.2.2.5

Elastic Shortening

The loss due to elastic shortening shall be calculated as follows.

a) Pretensioned Members:

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

b) Post-Tensioned Members:

$$ES = \left[\frac{N-1}{2N}\right]\frac{E_s}{E_{ci}} f_{cir}$$

9 — 9.2.3

Losses After Transfer

9 — 9.2.3.1

The losses after transfer shall be taken as:

$$\Delta f_{s2} = CR + SH + REL_2$$

9 — 9.2.3.2

Creep

a) Normal-Density Concrete:

Prestress loss due to creep of normal-density concrete shall be calculated as follows:

$$CR = [1.37 - 0.77\,(0.01H)^2]\,K_{cr}\left[\frac{E_s}{E_c}\right](f_{cir} - f_{cds})$$

where the term in the first brackets is a function of the mean annual relative humidity, H, expressed as a percentage and given in Appendix A2 — 1, $K_{cr} = 2$ for pretensioned members and $K_{cr} = 1.6$ for post-tensioned members.

b) Low-Density Concrete:

Creep loss in low-density concrete shall be based on the properties representative of the concrete to be used.

9 — 9.2.3.3

Shrinkage

Loss of prestress due to shrinkage shall be calculated as follows.

a) Normal-Density Concrete:

i) Pretensioned Members: $SH = 117 - 1.05\,H$

ii) Post-Tensioned Members: $SH = 94 - 0.85\,H$

b) Low-Density Concrete:

Shrinkage loss in low-density concrete shall be based on the properties representative of the concrete to be used.

9 — 9.2.3.4

Relaxation of Prestressing Steel

Loss of prestress due to relaxation after transfer shall be calculated as follows.

a) Stress-Relieved Strand and Wire:

$$REL_2 = \left[\frac{f_{si}}{f_{pu}} - 0.52\right]\left[0.42 - \frac{CR + SH}{1.25 f_{pu}}\right] f_{pu} \geq 0.01 f_{pu}$$

b) Low-Relaxation Strand and Wire:

$$REL_2 = \left[\frac{f_{si}}{f_{pu}} - 0.55\right]\left[0.34 - \frac{CR + SH}{1.25 f_{pu}}\right] \frac{f_{pu}}{3} \geq 0.002 f_{pu}$$

c) High-Strength Bars:
The relaxation loss, REL_2, in high-strength bars shall be based on approved test data.

Performance factors 9 — 10

The following values of performance factor ϕ shall be used:

- a) Flexural members with or without axial tension where the resistance is calculated in accordance with Clauses 9 — 11.1.1(a) and 9 — 11.1.2(a) or members subject to axial tension alone:
  - i) Cast-in-place members with depths up to and including 500 mm: 0.80
  - ii) Cast-in-place members with depths over 500 mm and precast members: 0.85
- b) Flexural members where the resistance is calculated in accordance with Clauses 9 — 11.1.1(b) and 9 — 11.1.2(b):
  - i) Cast-in-place members with depths up to and including 500 mm: 0.65
  - ii) Members with cast-in-place flanges where the flange thickness is 500 mm or less: 0.65
  - iii) Cast-in-place members with depths over 500 mm and precast members: 0.70
- c) Compression members: 0.65
- d) Compression with flexure:
  The value of ϕ may be increased linearly from the value for compression members to the value for flexural members as the factored axial load acting on a section at ultimate limit state decreases from $0.10A_g\ (0.85f'_c - 0.60f_{pc})$ to zero.
- e) Shear and torsion: 0.75

Proportioning of Sections for Flexure 9 — 11

General 9 — 11.1

The basic assumptions stated in Clause 9 — 6.2 shall be used in the calculation of the factored resistance of members. For typical sections, Clauses 9 — 11.1.1 and 9 — 11.1.2 may be used.

9 — 11.1.1 Rectangular sections or flanged sections with the neutral axis within the flange:

a) For members with reinforcement meeting the requirements of Clause 9 — 13.1.1(a):

$$M_u = \phi A_{ps} f_{ps} d \left[1 - 0.6\frac{\rho_p f_{ps}}{f'_c}\right]$$

b) For members with reinforcement in excess of the preferred amount specified in Clause 9 — 13.1.1(a):

$$M_u = \phi(0.25)f'_c bd^2$$

9 — 11.1.2 Flanged sections with the neutral axis within the web:

a) For members with reinforcement meeting the requirements of Clause 9 — 13.1.1(b):

$$M_u = \phi\left[A_{sr} f_{ps} d\left(1 - \frac{0.6A_{sr} f_{ps}}{b_w d f'_c}\right) + A_{sf} f_{ps}\,(d - 0.50\,h_f)\right]$$

where $A_{sr} = A_{ps} - A_{sf}$
and $A_{sf} = 0.85f'_c(b - b_w)\,h_f/f_{ps}$

b) For members with reinforcement in excess of the preferred amount specified in Clause 9 — 13.1.1(b):

$$M_u = \phi[0.25f'_c\,b_w d^2 + A_{sf} f_{ps}(d - 0.50\,h_f)]$$

where A_{sf} is defined in Clause 9 — 11.1.2(a).

Steel Stress 9 — 11.2

9 — 11.2.1

Bonded Tendons

The value of f_{ps} shall be computed using a method based on sectional strain compatibility and utilizing stress-strain curves representative of the steel, except that if $f_{se} \geq 0.5f_{pu}$, the value of f_{ps} may be computed from the following.

a) Stress-Relieved Strand and Wire:

$$f_{ps} = f_{pu}\left[1 - 0.5\frac{\rho_p f_{pu}}{f'_c}\right]$$

b) Low-Relaxation Strand and Wire:

$$f_{ps} = f_{pu}\left[1 - 0.4\frac{\rho_p f_{pu}}{f'_c}\right]$$

9 — 11.2.2 **Unbonded Tendons During Construction**

In lieu of a precise determination of f_{ps}, and provided $f_{se} \geq 0.5f_{pu}$, the value of f_{ps} may be computed from:

$$f_{ps} = f_{se} + 70 + 0.01\, f'_c/\rho_p$$

but not more than f_{py} nor $f_{se} + 400$

Non-Prestressed Reinforcement 9 — 12

General **9 — 12.1**

Non-prestressed reinforcement shall conform to the requirements of Section 8.

Contribution to Factored Resistance **9 — 12.2**

When used in combination with prestressed steel, non-prestressed reinforcement may be considered to contribute to the tensile force used to compute the factored resistance.

Shrinkage and Temperature **9 — 12.3**

For post-tensioned members, non-prestressed reinforcement to resist shrinkage and temperature effects shall be provided in directions both parallel to and normal to the axis of the structure. The ratio of non- prestressed reinforcement area to gross concrete area shall not be less than 0.0010 for slabs and 0.0015 for other sections. The spacing of such reinforcement shall not exceed 450 mm.

Maximum and Minimum Reinforcement 9 — 13

Maximum Reinforcement **9 — 13.1**

9 — 13.1.1

The preferred amount of prestressed and non-prestressed reinforcement shall be such that

a) For rectangular sections and flanged sections where the neutral axis lies within the flange.

$$\frac{A_{ps}f_{ps}}{bdf'_c} + \frac{A_s f_y}{bdf'_c} - \frac{A'_s f_y}{bdf'_c} \leq 0.30$$

b) For flanged sections where the neutral axis lies within the web.

$$\frac{A_{sr}f_{ps}}{b_w df'_c} + \frac{A_s f_y}{b_w df'_c} - \frac{A'_s f_y}{b_w df'_c} \leq 0.30$$

9 — 13.1.2

When the amount of reinforcement exceeds the preferred amount specified in Clause 9 — 13.1.1, the performance factors, ϕ, given in Clause 9 — 10(b) shall apply.

Minimum Reinforcement **9 — 13.2**

9 — 13.2.1

The total amount of prestressed and non-prestressed reinforcement shall be such that the calculated unfactored flexural resistance of the member is at least 1.25 times the cracking moment, where the cracking moment is defined as that which induces a tensile stress of $1.00f_r$ in the pre-stressed concrete section.

9 — 13.2.2

The requirements of Clause 9 — 13.2.1 may be waived if the reinforcement is proportioned to resist a factored moment one-third greater than that required by analysis.

Proportioning of Sections for Compression 9 — 14

General **9 — 14.1**

The proportioning of cross-sections subject to combined flexure and axial compression shall be based on the assumptions given in Clause 8 — 7.2 with the following additions:

a) Additional strain in the prestressing steel shall be assumed to be equal to that due to effective prestressing.

b) The stress in the prestressing steel shall be consistent with that indicated by the stress-strain curve of the material at the computed strain.

Slenderness Effects in Compression Members **9 — 14.2**

9 — 14.2.1 **General**

The proportioning of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall include the influence of axial loads and variable moment of inertia on member stiffness and moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads and prestressing forces.

9 — 14.2.2 **Approximate Evaluation of Slenderness Effect**

In lieu of the provisions given in Clause 9 — 14.2.1, the proportioning of compression members with slenderness ratios, $k\ell_u/r$, less than 100, may be based on the following approximate procedure:

a) Sections (a) to (e) of Clause 8 — 7.5.4.2 shall apply.

b) For eccentrically prestressed members, consideration shall be given to the effect of lateral deflection due to prestressing in determining the magnified moment.

c) Members shall be proportioned using the factored axial load, P, at the ultimate limit state, and a magnified moment, M_c, defined by:

$$M_c = \delta M_2$$

where: $$\delta = \frac{C_m}{1 - \dfrac{P}{\phi P_c}} \geq 1.0$$

and $$P_c = \frac{\pi^2 EI}{(k\,\ell_u)^2}$$

In lieu of a precise calculation, EI may be taken as:

$$EI = E_c I_g / \lambda$$

where: $\lambda = 17.5 - 25\,P/P_0 \geq 2.5$

The effect of sustained loading may be accounted for by dividing the value of EI by $(1 + \beta_d)$

For members braced against side-sway, and without transverse loads between supports for the loading case under consideration, C_m may be taken as:

$$C_m = 0.6 + 0.4 M_1/M_2$$

For all other cases, C_m shall be taken as 1.0.

d) Members shall be proportioned for an eccentricity equal to the greatest of:
   - i) that corresponding to the maximum moment which can accompany the compression, or
   - ii) $0.0025\,\ell_u$, or
   - iii) 0.10h

Proportioning of Sections for Shear and Torsion

9 — 15

General Reinforcement Requirements

9 — 15.1

9 — 15.1.1 **Shear Reinforcement — Minimum Requirements**

Shear reinforcement shall be provided in all flexural members except solid slabs. The area of shear reinforcement shall be as specified in Clause 8 — 8.2.2, except for members having $f_{se} \geq 0.40 f_{pu}$ where a minimum area of:

$$A_v = \frac{A_{ps}}{80} \frac{f_{pu}}{f_y} \frac{s}{d} \sqrt{\frac{d}{b_w}}$$

may be used.

9 — 15.1.2 **Shear Reinforcement — Details**

Detailing of the shear reinforcement shall be in accordance with Clause 8 — 9.6. However, unless restricted by Clause 8 — 8.2.13, the spacing of stirrups placed perpendicular to the axis of the member shall not exceed 0.75h, 3 b_w, or 600 mm, whichever is the smallest.

9 — 15.1.3 **Torsion Reinforcement — Minimum Requirements**

The minimum amount of reinforcement for torsion shall conform to the requirements of Clause 8 — 8.3.4.

9 — 15.1.4 **Torsion Reinforcement Details**

Detailing of torsion reinforcement shall be in accordance with Clause 8 — 9.7.

Shear Resistance

9 — 15.2

9 — 15.2.1 **General**

The factored shear resistance shall be computed from:

$$V_u = \phi(V_c + V_s)$$

where V_c shall be computed in accordance with Clause 9—15.3. Wherever applicable, the effects of inclined flexural compression in variable depth members may be included in the calculations. Axial tension due to restrained shrinkage and creep shall be considered. V_s shall be computed in accordance with Clause 8—8.2.11 and $(V_c + V_s)$ shall not exceed $0.25\ b_w d\ f'_c$.

9—15.2.2 Critical Section

When the reaction in the direction of the applied shear introduces compression into the end region of a member, sections located at a distance less than 0.5h from the face of a support may be proportioned for the shear force at 0.5h.

Shear Resistance Provided by Concrete **9—15.3**

9—15.3.1 For members having $f_{se} \geqslant 0.40\ f_{pu}$, unless a more detailed calculation is made in accordance with Clause 9—15.3.2, the shear resistance provided by concrete, V_c, shall be calculated from:

$$V_c = 0.27 f_r b_w d$$

9—15.3.2 The shear resistance provided by the concrete, V_c, shall not exceed the lesser of the shear resistance at flexural-shear cracking, V_{ci}, or web-shear cracking, V_{cw}, where:

a) The shear resistance at flexural-shear cracking shall be calculated from:

$$V_{ci} = 0.08\ f_r b_w d + V_d + V_\ell M_{cr}/M_{max}$$

but need not be taken less than $0.23\ f_r b_w d$.

The values of M_{max} and V_ℓ shall be computed from the load distribution causing maximum moment to occur at the section. M_{cr} shall be computed from:

$$M_{cr} = \frac{I}{y_t}(0.80 f_r + f_{pe} - f_d)$$

b) The shear resistance at web-shear cracking shall be calculated from:

$$V_{cw} = 0.47\ f_r b_w d + 0.3\ f_{pc} b_w d + V_p$$

Alternatively, V_{cw} may be taken as the shear corresponding to a multiple of dead load plus live load which results in a computed principal stress of $0.53\ f_r$ at the centroidal axis of the member, or at the intersection of the flange and the web when the centroidal axis is in the flange. In a composite member, the principal tensile stress shall be computed using the cross-section which resists live load.

In Clause 9—15.3, d shall be the distance from the extreme compression fibre to the centroid of the prestressing tendons or 0.80h, whichever is greater.

9—15.3.3 In a pretensioned member, in which the section at a distance 0.5h from the face of the support is closer to the end of the member than the transfer length of the tendons, the reduced prestress shall be considered when calculating V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length given by:

$$L_t = 1.50 \frac{f_{si}}{f'_{ci}} d_b - 117$$

Proportioning of Shear Reinforcement **9—15.4** Shear reinforcement shall be proportioned in accordance with Clauses 8—8.2.11 to 8—8.2.14. The effects of post-tensioned ducts in the web shall be considered in accordance with Clause 9—15.5.

Effect of Post-Tensioning Ducts in Webs **9—15.5** Where the web of a beam contains longitudinal post-tensioning ducts, the following equivalent web width, b_e, shall be used in computing the factored shear resistance:

a) Bonded Tendons:

$$b_e = b_w - \frac{2}{3}\Sigma d_d$$

b) Unbonded Tendons During Construction:

$$b_e = b_w - \Sigma d_d$$

In both cases Σd_d is the sum of all duct diameters at the level where b_e is a minimum. The sum of the duct diameters at any level shall not exceed $0.6\ b_w$.

Slabs and Footings **9—15.6** Slabs and footings which are reinforced in more than one direction shall be proportioned in accordance with Clause 8—8.8, except that when the average effective prestress exceeds 1.5 MPa, the shear resistance provided by the concrete, V_c, shall be taken as:

$$V_c = b_o d\,(0.40 f_r + 0.25 f_{pc}) + V_p$$

Members Subjected to Torsion **9—15.7** Members subjected to torsion shall be proportioned in accordance with Clause 8—8.3.

Alternative Methods of Proportioning for Bending, Shear and Torsion

9 — 15.8

In lieu of the procedure described in Clauses 9 — 15.2 to 9 — 15.7, the resistance of a prestressed concrete member may be determined by an experimentally-verified rational procedure which takes into account:

a) the requirements of equilibrium for the section,
b) the requirements of geometric compatibility, and
c) the stress-strain characteristics of the steel and of the cracked concrete.

Development Length of Pretensioning Strand

9 — 16

Pretensioning strand shall be bonded beyond the critical section for a development length not less than:

$$L_d = 1.5\frac{f_{si}}{f'_{ci}}d_b - 117 + 0.18(f_{ps} - f_{se})d_b$$

in which the constant 0.18 has the dimension of 1/MPa.

Where bonding of the strand does not extend to the end of the member and tension occurs in the precompressed tensile zone at serviceability limit states Type II, a development length of $2L_d$ shall be used.

Anchorage Zones, Couplers and Ducts

9 — 17

Anchorage Zone Reinforcement

9 — 17.1

9 — 17.1.1 **General**

Anchorage zones shall be reinforced to resist tensile, bursting and spalling forces induced by concentrated loads due to prestressing.

9 — 17.1.2 **Post-Tensioning**

9 — 17.1.2.1 Provision shall be made for the distribution of concentrated prestressing forces.

9 — 17.1.2.2 The maximum bursting stress, f_{bs}, shall be computed from:

$$f_{bs} = \psi F_{sj}/a_2^2$$

and shall not exceed $(0.80f_{n} + 20\rho_{bs})$, where $\psi = e^{-3a_1/a_2}$

When calculating the maximum bursting stress, a symmetrically placed square anchor of side a_1 acting on a square prism of side and depth a_2 shall be assumed. The dimension a_2 shall be the minimum distance between the centreline of anchors or 2 times the distance from the centreline of the anchor to the nearest edge of concrete, whichever is lesser. For circular anchors, a_1 shall be taken as the side of a square with an area equal to the area of the circular anchor.

9 — 17.1.2.3 The total bursting force, F_{bs}, in a plane perpendicular to the longitudinal axis of the tendon shall be computed from:

$$F_{bs} = 0.70F_{sj}\psi$$

Reinforcement to resist the bursting force shall be uniformly distributed from $0.52X_m$ to a distance equal to a_2, measured from the loaded face of the end block, where:

$$X_m = 0.54(1 - \psi)a_2$$

The stress in the reinforcement shall not exceed 200 MPa. This reinforcement shall be anchored in accordance with Clause 8 — 10.

9 — 17.1.2.4 Reinforcement to control spalling cracks in both the horizontal and vertical planes at the anchorage zones shall be provided within 0.2h of the end of the member. The end stirrup shall be placed as close to the end of the member as practicable. The reinforcement shall extend over the full depth and width of the member and shall be anchored in accordance with Clause 8 — 10. The stress in the reinforcement shall not exceed 140 MPa.

9 — 17.1.2.5 The anchorage zone reinforcement shall be detailed to allow compaction and vibration of concrete around the anchorages. Air vents and grout tubes shall be kept away from zones of high bearing stress.

9 — 17.1.3 **Pretensioning**

9 — 17.1.3.1 End blocks are not required where all tendons are pretensioned strand.

9 — 17.1.3.2 Vertical stirrups to resist at least 4% of the prestressing force at transfer shall be distributed uniformly over a length equal to 0.2h from the end of the girder. The end stirrup shall be placed as close to the end of the member as practicable. The stress in the reinforcement shall not exceed 140 MPa.

9 — 17.1.3.3 The ends of members with flanges shall be reinforced to enclose the prestressing steel in the flanges. The size and spacing of this reinforcement shall be in accordance with Clause 9 — 17.1.3.2.

Transverse reinforcement shall be provided in the flanges of box girders and shall be anchored into the webs of the girder.

Detailing Requirements of Post-Tensioning Anchorages and Couplers **9—17.2**

9—17.2.1 Anchorages and end fittings shall be protected against corrosion. The concrete cover shall conform with the requirements for reinforcement specified in Clause 8—9.3.

9—17.2.2 Couplers shall not be located at points of sharp tendon curvature nor in the vicinity of points of maximum moments. Couplers and their components shall be enclosed in housings. The housings shall be long enough to permit the necessary movements, and shall be provided with fittings to allow complete grouting.

Post-Tensioning Ducts **9—17.3**

9—17.3.1 Ducts shall be grouted after post-tensioning.

9—17.3.2 Sheaths for post-tensioning ducts shall be made of bright steel or other approved materials. Sheaths shall be corrugated, watertight under an internal pressure 350 kPa, and chemically non-reactive with concrete, tendons, or grout.

9—17.3.3 The inside diameter of the sheath shall be at least 6 mm larger than the nominal diameter of single wire, bar or strand tendons; or in the case of multiple wire, bar or strand tendons, the inside cross-sectional area of the sheath shall be at least 2 times the area of the prestressing steel.

9—17.3.4 Rigid steel sheaths shall have a wall thickness of at least 0.60 mm and shall be capable of being bent to a minimum inside radius of 9 m without distress.

9—17.3.5 Semi-rigid steel sheaths shall have a wall thickness of at least 0.25 mm and shall be capable of being bent to a minimum inside radius of 3.50 m without distress.

Concrete Protection for Reinforcement 9—18

Prestressing Steel **9—18.1**

9—18.1.1 The minimum concrete cover shall be as follows.

a) Pretensioning Strands:
  i) tops of decks: 75 mm
  ii) elsewhere: 40 mm
b) Post-Tensioning Ducts
  i) tops of decks:
    longitudinal ducts: 115 mm
    transverse ducts with $d_d \leq 55$ mm: 75 mm
    $d_d > 55$ mm: 115 mm
  ii) elsewhere: 40 mm or $0.50d_d$, whichever is greater
  iii) members cast against earth: 75 mm

9—18.1.2 The average concrete cover which allows for tolerances in placing the prestressing steel shall be used in proportioning the components. Unless otherwise approved, the tolerances shall be as follows.

a) Pretensioning Strands
  i) tops of decks: ± 5 mm
  ii) elsewhere: ± 5 mm
b) Post-Tensioning Ducts
  i) tops of decks:
    longitudinal ducts: ± 15 mm
    transverse ducts, $d_d \leq 55$ mm: ± 10 mm
    $d_d > 55$ mm: ± 15 mm
  ii) elsewhere: ± 15 mm
  iii) members cast against earth: ± 15 mm

9—18.1.3 The average concrete covers and the permissible tolerances shall be shown on the plans.

Non-Prestressed Reinforcement **9—18.2** The minimum concrete cover to non-prestressed reinforcement shall be in accordance with Clause 8—9.3.

Spacing of Strands and Ducts 9—19

Pretensioning Strands **9 — 19.1**

The minimum clear distance between strands at the end of members shall not be less than 2.75 times the nominal diameter of the strands or 1.33 times the maximum size of the coarse aggregate, whichever is greater.

Post-Tensioning Ducts **9 — 19.2**

9 — 19.2.1

The minimum clear distance between ducts shall not be less than 1.33 times the maximum size of the coarse aggregate or 25 mm, whichever is greater.

9 — 19.2.2

For groups of ducts where the ducts are located in a single horizontal plane, the minimum clear horizontal distance between each group shall not be less than 100 mm. A group shall contain no more than three ducts.

For groups of ducts where the ducts are located in two or more horizontal planes, the minimum clear horizontal distance between adjacent groups shall not be less than 100 mm. A group shall contain no more than two ducts in the same horizontal plane.

For precast or cast-in-place segmental construction, the minimum clear horizontal distance specified in this clause may be reduced to 75 mm.

Bundling **9 — 19.3**

Pretensioning strands may be bundled, provided that the spacing specified in Clause 9 — 19.1 is maintained for a distance from the end of the member equal to the minimum development length given in Clause 9 — 16. Groups of 8 strands of Size Designation 13 or smaller, or 5 strands larger than Size Designation 13, may be bundled to touch one another in a vertical plane. The number of strands bundled in any other manner shall not exceed four. The minimum clear distance between groups of bundled strands shall not be less than 1.33 times the maximum size of the aggregate or 25 mm, whichever is greater.

Directional Change in Prestressing Steel **9 — 19.4**

Thrusts produced by directional change in prestressing steel shall be investigated and resisted by compression in the concrete or by non-prestressed reinforcement. The stress in the non-prestressed reinforcement shall not exceed $0.60f_y$ or 200 MPa, whichever is smaller.

Deformations **9 — 20**

General **9 — 20.1**

Deflections, rotations and camber due to prestress, and other loads shall be considered in design. Deformations shall be computed by elastic analysis using the gross concrete section, and shall include the effects of prestress, other permanent loads, prestress losses, creep and shrinkage.

Deflections **9 — 20.2**

In lieu of a detailed analysis, long-term camber as a function of instantaneous camber for members constructed and prestressed in a single stage, may be computed as follows:

$$\delta_u = \delta_i \left\{ 1 + 0.145CR \frac{E_c}{E_s} \left[1 - \left(0.145CR \frac{E_c}{E_s} + \frac{SH}{ES} \right) (1 - \gamma) \frac{\gamma}{2} \right] \right\} \left\{ 1 - \frac{\Delta f_{s2}}{f_{si}} \right\}$$

where: $\gamma = \frac{E_s}{E_c} \frac{A_{ps}}{A_g} \left[1 + \left(\frac{e}{r} \right)^2 \right]$

and the constant 0.145 has the dimension of 1/MPa.

The camber δ_t at anytime, t days after loading may be calculated from:

$$\delta_t = \delta_i + (\delta_u - \delta_i)(1 - e^{-0.08 \cdot \sqrt{t}})$$

In computing the camber of a member at any time, the instantaneous deflections due to loads applied prior to that time shall be algebraically added to δ_t.

Statically-Indeterminate Structures **9 — 21**

General **9 — 21.1**

For statically-indeterminate structures the moments, shears, reactions and thrusts produced by prestress and other loads shall be determined by elastic analysis.

Secondary Effects **9 — 21.2**

The secondary effects induced by prestress shall be included in calculations at serviceability limit states Type II. In investigating the resistance of sections at ultimate limit states, the factored secondary effects shall be added algebraically to factored load effects.

Effect of Creep **9 — 21.3**

The effects of redistribution of dead load and prestressing effects due to creep shall be included in the design of statically-indeterminate structures constructed in a configuration different from the final one.

Composite Semi-Continuous Structures **9 — 21.4**

9 — 21.4.1 **General**

This clause governs the design of superstructures of two or more spans composed of precast, prestressed, single span units with cast-in-place composite deck slabs, and made continuous for live load and superimposed dead load by providing non-prestressed reinforcement in the deck slab in the negative moment regions.

The provisions of Clauses 9—15 and 9—22 shall apply.

9—21.4.2 **Positive Moments**

9—21.4.2.1 Provision shall be made for positive moments which may develop at serviceability limit states Type II due to creep and shrinkage, temperature, and live load. Deformation and settlement of the piers shall be considered.

9—21.4.2.2

Positive moment resistance at a pier may be developed by providing non-prestressed reinforcement adequately embedded in the bottom flange of the beam and anchored into the diaphragm over the pier. The stress in the non-prestressed reinforcement for positive moment connection shall not exceed $0.60f_y$ or 250 MPa, whichever is smaller.

9—21.4.3 **Negative Moments**

The reinforcement in negative moment regions shall be proportioned in accordance with Section 8. The effect of precompression due to prestress in the girders may be neglected in computing the resistance if the maximum precompression stress after all losses is less than $0.45f'_c$ and the continuity reinforcement ratio, ρ, in the deck slab is less than 0.015. The factored resistance shall be calculated using the specified strength of the girder concrete.

Composite Construction 9—22

General **9—22.1**

9—22.1.1 The requirements of Clause 9—22 apply to flexural members consisting of precast prestressed concrete units so interconnected with cast-in-place concrete that they act together as an integral unit.

9—22.1.2 Precast concrete units shall be proportioned to support all loads applied prior to the cast-in-place concrete attaining a strength of $0.75f'_c$. The applied loads shall be factored in accordance with Section 2.

Flexure **9—22.2**

9—22.2.1 When the components of the composite section have different specified strengths, a linear strain distribution shall be assumed, and stresses shall be computed on the basis of the respective moduli of elasticity.

9—22.2.2 The flexural stresses at serviceability limit states Type II shall be computed from the superposition of stresses appropriate to each stage of loading.

9—22.2.3 Differential shrinkage between cast-in-place concrete and precast units shall be considered in the design of composite members at serviceability limit states Type II.

9—22.2.4 The factored resistance of a composite section shall be computed in the same manner as for a monolithically cast unit.

Shear **9—22.3**

9—22.3.1 The factored resistance of a composite section shall be based on its total depth and shall be computed in accordance with Clause 9—15.

9—22.3.2 Interface shear shall be investigated and provided for in accordance with Clause 8—8.7.

T-Girders 9—23

General **9—23.1**

9—23.1.1 Composite T-girders consisting of precast beams and cast-in-place slabs shall satisfy the requirements of Clause 9—22.

9—23.1.2 The interface shear at the junction of the flange and web for cast-in-place or precast T-girders shall be provided for in accordance with Clause 8—8.7.

Effective Flange Width **9—23.2** The effective flange width to be used in the design shall be in accordance with the requirements of Section 3.

Flange Thickness **9—23.3** The thickness of the top flange shall be sufficient to resist flexure and shall not be less than 0.06 times the clear distance between the webs nor less than 225 mm for cast-in-place girders and 140 mm for precast girders.

Isolated Girders **9—23.4** Isolated girders in which the T-form is used for the purpose of providing additional compression area, shall have a flange thickness at least equal to 0.33 times the web width or 100 mm, whichever is greater. The flange width shall not exceed:

a) four times the web width in composite construction, and

			b) nine times the web width in monolithic construction.
	Diaphragms	**9 — 23.5**	Diaphragms shall be provided in accordance with Clause 8 — 12.5.
Box Girders		**9 — 24**	
	General	**9 — 24.1**	
		9 — 24.1.1	Composite girders consisting of precast box beams and cast-in-place slabs shall satisfy the requirements of Clause 9 — 22.
		9 — 24.1.2	The interface shear at the junction of the flange and webs of precast or cast-in-place box girders shall be investigated and provided for in accordance with Clause 8 — 8.7.
	Effective Flange Width	**9 — 24.2**	The effective flange width to be used in the design shall be in accordance with the requirements of Section 3.
	Flange Thicknesses	**9 — 24.3**	
		9 — 24.3.1	**Top Flange** The thickness of the top flange shall be sufficient to resist flexure in the transverse direction and shall not be less than 0.06 times the clear distance between the webs nor less than 225 mm for cast-in-place girders and 140 mm for precast girders.
		9 — 24.3.2	**Bottom Flange** The thickness of the bottom flange shall not be less than 0.06 times the clear distance between the webs nor less than 150 mm for cast-in-place girders and 140 mm for precast girders. Fillets shall be provided at the intersections of all surfaces within the cell of a box girder except at the junction of web and bottom flange where they may be omitted.
	Top and Bottom Flange Reinforcement	**9 — 24.4**	The minimum reinforcement in top and bottom flanges of cast-in-place post-tensioned box girders shall be in accordance with Clause 8 — 13.3.3 except in the bottom flanges, where the total area of non-prestressed reinforcement parallel to girder span shall not be less than 0.003 times the flange area.
	Diaphragms	**9 — 24.5**	Diaphragms shall be provided in accordance with Clause 8 — 13.4.
Multi-Beam Decks		**9 — 25**	Multi-beam decks consisting of precast units placed side by side shall be provided with a reinforced concrete structural slab of at least 125 mm thickness capable of providing shear transfer between the units. The transverse shear due to live load shall be computed in accordance with Section 3, and the concrete slab shall be reinforced in accordance with Clause 8 — 8.4.
Segmental Girders		**9 — 26**	
	General	**9 — 26.1**	The provisions of Clause 9 — 26 apply to prestressed concrete box girders made of match-cast precast or cast-in-place segments and which are erected by cantilevering from the piers.
	Design of Superstructure	**9 — 26.2**	
		9 — 26.2.1	**General** The design of segmental girders shall include the effects of the anticipated method of construction and associated construction loads.
		9 — 26.2.2	**Shear Keys** Shear keys shall be provided at joints between precast segments and shall be proportioned to transfer shear forces during erection. The shear resistance provided by the concrete under factored loads at the time of erection shall be based on a shear stress of $0.36f_r$.
		9 — 26.2.3	**Deflections** Deflections shall be calculated at the time of design, and shall be based on an anticipated production and/or erection schedule. The plans shall indicate that compensation is required for such deflection during manufacture or construction of segments. During construction the actual deflections shall be monitored and compared with the calculated deflections, and, if necessary, adjustments shall be made to subsequent segments as construction proceeds.
		9 — 26.2.4	**Diaphragms** Diaphragms shall be provided at abutments and piers and shall permit transfer of shear from webs to the bearings.
		9 — 26.2.5	**Joint Details** Epoxy joint filler shall be used between match-cast precast segments. The minimum thickness of epoxy shall be 2 mm on each surface if applied to both surfaces, or 3 mm if applied to one surface.

A minimum compressive stress of 0.35 MPa shall be provided over the entire joint area between precast segments by means of temporary post-tensioning until the permanent tendons are fully stressed.

Design of Substructure **9—26.3**

Unbalanced moments due to segment weights and construction loads shall be provided for in the design of the piers, or shall be accommodated by the use of temporary supports or other methods.

Piles 9—27

General **9—27.1**

The design of prestressed concrete piles shall conform to the requirements of Sections 6 and 9 except as modified in Clause 9—27.

Specified Concrete Strength **9—27.2**

The minimum specified strength of concrete shall be 35 MPa.

Effective Prestress **9—27.3**

Prestressing steel shall be placed and stressed to provide a minimum effective prestress of 3 MPa for piles up to 12 m long and 5 MPa for piles longer than 12 m. The effective prestress shall not exceed $0.20f'_c$.

Handling **9—27.4**

Stresses during handling and transportation shall be investigated, taking into account the dead load of the pile plus a 50% allowance for dynamic effects.

Cross-Sections **9—27.5**

Prestressed concrete piles may be solid or hollow. The minimum wall thickness for hollow piles shall be 125 mm.

Concrete Stress Limitations **9—27.6**

9—27.6.1 **Handling**

The stresses during handling shall not exceed:

a) Compression: $0.06f'_c$

b) Tension: $0.80f_r$

9—27.6.2 **Under Loads**

The stresses at serviceability limit states Type II loads acting on a pile shall be such that:

a) $\left(f_{pe} + P_{ser}/A_g + \dfrac{M_{ser}\,c}{I}\right) \leq$ zero for tension or $\leq 0.45f'_c$ for compression

b) $(P_{ser}/P_a + M_{ser}/M_a) \leq 1$

where: $P_a = (0.33f'_c - 0.27f_{pc})A_g$, for laterally-supported piles

$P_a = R(0.33f'_c - 0.27f_{pc})A_g$, for laterally-unsupported piles with $\ell_e/r \leq 120$

$\ell_e = 1.0\,\ell$ for piles hinged at both ends

$\ell_e = 0.8\,\ell$ for piles fixed at one end

$\ell_e = 0.6\,\ell$ for piles fixed at both ends

A_g = cross-sectional area of pile

$R = (1.23 - 0.008\ell_e/r) \leq 1.0$

$M_a = f_{pc}(I/c)$

Factored Resistance **9—27.7**

9—27.7.1 The basic assumptions given in Clause 9—6.2 shall be used in the computation of the factored resistance of piles. Slenderness effects as given in Clause 9—14.2 shall be included.

9—27.7.2 The factored resistance of a laterally-supported pile in compression may be computed from:

$$P_u = \phi(0.85f'_c - 0.60f_{pc})A_g$$

9—27.7.3 The factored resistance of a pile in flexure may be computed from:

$$M_u = \phi K_1 h\, A_{ps} f_{pu}$$

where K_1 is: 0.37 for solid square piles;

0.32 for solid circular, octagonal, and hexagonal piles;

0.38 for hollow square piles; and

0.34 for hollow circular and octagonal piles.

9—27.7.4 The factored resistance of a prestressed concrete pile shall include the interaction of compression and flexure.

Sections Within Development Length **9—27.8**

The effect of the transfer length on the stresses at serviceability limit states Type II, and the development length on the factored resistance, shall be investigated.

Reinforcement Details **9—27.9**

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9 — 27.9.1 The full length of prestressing steel shall be enclosed with spiral reinforcement meeting the requirements of CSA Standard G30.3, *Cold-Drawn Steel Wire for Concrete Reinforcement*.

9 — 27.9.2 For piles up to 600 mm in diameter, spiral wire shall be at least 5 mm diameter. Spiral reinforcement at the ends of these piles shall have a pitch of 75 mm for 16 turns. In addition, the top 150 mm of pile shall have 5 turns of spiral winding at 25 mm pitch. For the remainder of the pile, the prestressing steel shall be enclosed with spiral reinforcement with not more than 150 mm pitch.

9 — 27.9.3 For piles having a diameter greater than 600 mm, spiral wire shall be at least 6 mm diameter. Spiral reinforcement at the ends of these piles shall have a pitch of 50 mm for 16 turns. In addition, the top 150 mm of pile shall have 4 turns of spiral winding at 40 mm pitch. For the remainder of the pile, the prestressing steel shall be enclosed with spiral reinforcement with not more than 100 mm pitch.

Concrete Protection

9 — 27.10 The minimum concrete cover to reinforcement shall be in accordance with Clause 9 — 18.

Splices

9 — 27.11

9 — 27.11.1 The resistance of a splice shall be at least equal to that of the pile.

9 — 27.11.2 The shape and size of splices shall be such as not to affect the performance of the pile.

9 — 27.11.3 The slack in mechanical splices shall be less than 0.50 mm in either compression or tension.

Poles

9 — 28

General

9 — 28.1 The design of prestressed concrete poles shall be in accordance with the requirements of Sections 5 and 9 except as modified in Clause 9 — 28.

Specified Concrete Strength

9 — 28.2 The minimum specified strength of concrete shall be 35 MPa.

Handling

9 — 28.3 The stresses during handling and transportation shall be investigated, taking into account the dead load of the pole plus a 50% allowance for dynamic effects.

Concrete Stress Limitations

9 — 28.4

9 — 28.4.1 **Handling**

The stresses during handling shall not exceed:

a) Compression: $0.60f'_c$

b) Tension: $1.00f_r$

9 — 28.4.2 **Under Loads**

The stresses at serviceability limit states Type II loads shall not exceed:

a) Compression: $0.45f'_c$

b) Tension: $0.80f_r$

Factored Resistance

9 — 28.5

9 — 28.5.1 The basic assumptions given in Clause 9 — 6.2 shall be used in the computation of the factored resistance of poles. The effect of lateral deflections shall be included.

9 — 28.5.2 In lieu of a detailed analysis, the factored resistance of hollow circular poles may be computed from:

a) Flexure:

$$M_u = \phi 0.34\, h\, A_{ps} f_{pu}$$

b) Torsion:

$$T_u = 2\phi A_o \sqrt{\frac{A_t f_y}{s} \frac{A_\ell f_{\ell y}}{p_o}}$$

9 — 28.5.3 The resistance of prestressed concrete poles subject to combined flexure and torsion shall be such that:

$$\frac{M}{M_u} + \left(\frac{T}{T_u}\right)^2 \leq 1.0$$

Deflection

9 — 28.6 The long-term lateral deflection at the top of the pole under dead load shall not exceed 1% of the pole height above the ground.

Circumferential Reinforcement

9 — 28.7 The minimum circumferential reinforcement shall be in accordance with Clause 8 — 17.6.3.

Concrete Protection **9—28.8**

9—28.8.1 The minimum concrete cover to reinforcement shall be in accordance with Clause 9—18.

9—28.8.2 Any chairs or other devices used between the reinforcement and the forms shall be made of non-corrosive material and shall not deface nor weaken the poles.

Resistance To Impact **9—28.9** Poles exposed to traffic shall be proportioned so that the failure due to impact from vehicular loads is ductile. This ductility may be achieved by providing at least 0.33 of the flexural resistance by non-prestressed reinforcement in the lower portion of the pole. The reinforcement shall be anchored below the ground level, and shall be evenly distributed over the cross-section.

10 Structural Steel

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Reference Publications

Section 10 makes reference to the following publications:

CSA Standards	G28-1968	*Carbon Steel Castings for General Application*
	CAN3-G40.20-M81	*General Requirements for Rolled or Welded Structural Quality Steel*
	CAN3-G40.21-M81	*Structural Quality Steels*
	G189-1966	*Sprayed Metal Coatings for Atmospheric Corrosion Protection*
	CAN3-S16.1M-78	*Steel Structures for Buildings — Limit States Design*
	W47.1-1983	*Certification of Companies for Fusion Welding of Steel Structures*
	W48.1-M1980	*Mild Steel Covered Arc-Welding Electrodes*
	W48.3-M1982	*Low Alloy Steel Arc-Welding Electrodes*
	W48.4-M1980	*Solid Mild Steel Electrodes for Gas Metal Arc-Welding*
	W48.5-M1982	*Mild Steel Electrodes for Flux Cored Arc-Welding*
	W48.6-M1980	*Bare Mild Steel Electrodes and Fluxes for Submerged Arc-Welding*
	W59 1982	*Welded Steel Construction (Metal Arc-Welding)*
ASTM Standards	A108-73	*Steel Bars*
	A325-76	*High Strength Bolts for Structural Steel*
	A370-77	*Standard Methods and Definitions for Mechanical Testing of Steel Products*
	A486-74	*Steel Castings for Highway Bridges*
	A490-70	*Quenched and Tempered Alloy Steel Bolts*

Scope

10—1

The design of those components not covered in this section shall be subject to approval.

Definitions

10—2

Buckling Load — the load at which a member reaches a condition of instability.

Brittle Fracture — a type of fracture in structural materials that usually occurs suddenly and without prior plastic deformation.

Composite Beam — a homogeneous steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Critical Net Area — that area which results in the least tensile resistance.

Erection Tolerances — the tolerances related to the plumbness, alignment, or level of a component with respect to the positions stipulated on the drawings.

Hybrid Beam — a steel beam composed of a web and a flange or flanges which are not all of the same specified yield strength.

Lateral or Lateral Torsional Buckling — buckling of a member involving lateral deflection and/or twist.

Local Buckling — the buckling of a plate element subject to compression influenced by the slenderness of the element, boundary and loading conditions, and the yield strength of the steel, and distinct from the instability of the member as a whole.

Notch Toughness — the ability of steel to absorb energy in the presence of a notch.

Post-Buckling Resistance — the phenomenon by which plate elements resist additional load after initial elastic buckling.

Slenderness Ratio — the effective length of a member divided by the corresponding radius of gyration.

Tension-Field Action — the truss-like behaviour of a plate girder panel under shear force characterised by diagonal tensile forces in the web and compressive forces in the transverse stiffeners.

Web Crippling — the local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

Notation

10—3

A	area, mm^2.
A'	area enclosed by the median line of a wall, mm^2.
A_b	area of a bolt, mm^2.
A_{cf}	area of compression flange of a steel section, mm^2.
A_c	area of concrete in a tube pile, mm^2.
A_f	area of bottom flange of box girders including longitudinal stiffeners, mm^2; or area of flanges of plate girder, mm^2.
A_g	gross area, mm^2.
A_m	area of fusion face, mm^2.
A_n	critical net area, mm^2; or total net area of a member tributary to the particular lap splice including elements not directly connected, mm^2.
A'_n	reduced net area of tension members accounting for shear lag effects, mm^2.
A_s	area of steel section, mm^2; or area of stiffener or part of stiffeners, mm^2.
A'_s	area of reinforcing steel within the effective width of a concrete slab, mm^2.
A_{tf}	area of top flange of steel section, mm^2.
A_w	web area, mm^2; or shear area, mm^2; or effective throat area of weld, mm^2.
a	spacing of transverse stiffeners, mm; or depth of compression block in a concrete slab, mm; or distance between centroids of groups of fasteners or welds, mm.
B_r	factored bearing resistance of a member or component, N.
b	0.5 of width of flange of beams and tees, mm; or full width of flange of channels and zed sections, and stems of tees, mm; or distance from free edge of plates to the first line of bolts or welds, mm; or width of stiffener, mm; or width of bottom flange plate between webs of box girder, mm.
b_c	effective width of concrete slab, mm.
b_s	width of compressive flange between longitudinal stiffeners, mm; or distance from web to nearest longitudinal stiffener, mm.
C	factored compressive resistance, N; or coefficient in stiffener of area formula.
C'	reduced factored compressive resistance of concrete, N.

C_c	factored compressive resistance of concrete, N.
C_e	Euler buckling force, N.
C_f	compressive force in a member or component at ultimate limit state, N.
C_r	factored compressive resistance of a member or component, N; or factored compressive resistance of steel acting at the centroid of the part of the steel area in compression, N.
C_r'	factored compressive resistance of concrete area, A_c, of the column, N.
C_{rc}	factored compressive resistance of a composite tube pile, N.
C_s	factored compressive resistance of reinforcing steel, N.
C_w	warping torsional constant, mm^6.
C_y	axial compressive force at yield stress, N.
c	coefficient in buckling stress formula.
D	stiffener factor; or outside diameter of circular section, mm; or diameter of rocker or roller, mm.
D_c	depth of compressive portion of web, mm.
d	depth, mm; or diameter of bolt or stud, mm; or longitudinal distance centre-to-centre of battens, mm.
d'	reduced depth of steel section, mm.
d_b	depth of beam, mm.
d_o	distance between transverse stiffeners, mm.
d_s'	distance from extreme compression fibre to centroid of reinforcing steel, mm.
E	modulus of elasticity of steel, MPa.
E_c	modulus of elasticity of concrete, MPa.
e	edge distance, mm.
e_c	moment arm, mm.
e_s	moment arm, mm.
F_c	factored compressive stress, MPa.
F_{cr}	buckling stress, MPa.
F_s	ultimate shear stress, MPa.
F_{st}	force in stiffener at ultimate limit state, N.
F_u	specified minimum tensile strength, MPa.
F_y	specified minimum yield stress, yield point, or yield strength, MPa.
F_{yc}	yield point of a column, MPa.
F_{yf}	specified minimum yield stress of the higher strength flange material in a hybrid girder, MPa.
f_c'	specified compressive strength of concrete, MPa.
f_y'	specified minimum yield stress of reinforcing steel, MPa.
G	shear modulus of steel, MPa.
H	height of stud, mm.
g	transverse spacing between fastener gauge lines, mm.
h	clear depth of web between flanges, mm; or width of rectangular hollow section, mm.
h_c	clear depth of column web, mm.
h_p	depth of sub-panel of a girder, mm.
I	moment of inertia, mm^4.
I_s	moment of inertia of longitudinal compression flange stiffener, mm^4.
I_t	moment of inertia of transverse compression flange stiffener, mm^4.
J	St. Venant's torsion constant.
j	coefficient used in determining moment of inertia of stiffeners.
K	effective length factor.
k	distance from outer face of flange to toe of flange-to-web fillet, mm.
k_v	shear buckling coefficient.
k_1	buckling coefficient in box girders.
k_2	buckling coefficient in box girders.
L	length, mm; or connection length in direction of loading equal to the distance between first and last bolts in bolted connections and to the overall length of the weld pattern in welded connections, mm; or the laterally-unsupported distance from one braced location to an adjacent braced location, mm; or length of roller or rocker, mm; or length of cut-out in closed cross-section member measured parallel to the longitudinal axis of the member, mm.

l	length in which warping restraint is developed, mm.
M_d	bending moment in a beam or girder at ultimate limit state due to dead load, N.
M_f	bending moment in a member or component at ultimate limit state, N·mm.
M_{ff}	lateral bending moment in flanges caused by torsion at ultimate limit state, N·mm.
M_{fr}	bending moment in a longitudinal rib at ultimate limit state, N·mm.
M_{fx}	bending moment in a member or component about the x-axis of the cross-section at ultimate limit state, N·mm.
M_{fy}	bending moment in a member or component about the y-axis of the cross-section at ultimate limit state, N·mm.
M_{f1}	smaller end moment of a beam column at ultimate limit state, N·mm.
M_{f2}	larger end moment of a beam column at ultimate limit state, N·mm.
M_{ll}	bending moment in a beam or girder at ultimate limit state due to live load, N·mm.
M_p	plastic moment = ZF_y, N·mm.
M_r	factored moment resistance of a member or component, N·mm.
M_r'	reduced moment resistance of a member or component, N·mm.
M_{rf}	factored moment resistance of a flange, N·mm.
M_{rp}	factored moment resistance of deck plate, N·mm.
M_{rr}	factored moment resistance of a longitudinal rib, N·mm.
M_{rx}	factored moment resistance of a member or component about the x-axis of the cross-section, N·mm.
M_{ry}	factored moment resistance of a member or component about the y-axis of the cross-section, N·mm.
M_{sdl}	bending moment in a beam or girder at ultimate limit state due to superimposed dead load, N·mm.
M_u	moment resistance of a member subject to lateral buckling, N.
M_y	yield moment, N·mm.
M_1	smaller moment at end of unbraced length of beam at ultimate limit state, N·mm.
M_2	larger moment at end of unbraced length of beam at ultimate limit state, N·mm.
m	number of faying surfaces or shear planes in a bolted joint, equal to 1 for bolts in single shear and 2 for bolts in double shear.
N	length of bearing of an applied load, mm; or number of shear connectors.
n	number of longitudinal stiffeners in box girders; or number of parallel planes of battens; or number of bolts; or modular ratio, E/E_c.
P	unfactored axial resistance of reinforced concrete slab, N.
Q_f	torsional moment in a member at ultimate limit state, N·mm.
Q_r	factored torsional resistance, N·mm; or resistance of shear connectors, N.
Q_{r1}	factored torsional resistance due to St.Venant torsion, N·mm.
Q_{r2}	factored torsional resistance due to warping restraint, N·mm.
R	coefficient used in moment resistances of hybrid girders; or radius of curvature of girder web, mm.
R_s	vertical force applied at longitudinal to transverse stiffener connection in box girders, at ultimate limit state, N.
R_w	vertical force applied at transverse stiffener to web connection in box girders, at ultimate limit state, N.
r	radius of gyration, mm.
r_t	radius of gyration about the axis of symmetry of a T-section comprising the compression flange and 1/6 of the web, mm.
r_y	radius of gyration of a member about its weak axis, mm.
S	elastic section modulus of a steel section, mm^3.
S'	elastic section modulus comprising the steel section and reinforcement within the effective width of the slab, mm^3.
S_b	elastic section modulus of the section referred to the bottom flange, mm^3.
S_{tt}	section modulus of a longitudinal stiffener with respect to the tip of the stiffener, mm^3.
S_n	elastic section modulus comprising the steel section and the composite concrete slab computed using the modular ratio, n, mm^3.

S_{3n}	elastic section modulus comprising the steel section and the composite concrete slab computed using a modular ratio of 3 x n, mm^3.
S_p	section modulus of a deck plate based on the effective width of the plate, mm^3.
S_t	section modulus of a transverse stiffener, mm^3.
s	centre-to-centre spacing between successive fastener holes in the line of load, mm.
T	tension in a bolt at serviceability limit state, N.
T_f	tensile force in a member or component at ultimate limit state, N.
T_r	factored tensile resistance of a member or component, N.
t	thickness, mm.
t_b	thickness of beam flange, mm.
t_c	thickness of concrete slab, mm; or thickness of column flange, mm.
t_{tf}	thickness of top flange, mm.
V	shear in a bolt or bolts at serviceability limit state, N.
V_f	shear force at ultimate limit state, N.
V_r	factored shear resistance of a member or component, N.
V_s	slip resistance at serviceability limit state Type II, N.
w	web thickness, mm.
w_c	thickness of column web, mm.
$\bar{x}$	distance perpendicular to direction of loading between centre of gravity of area A_n and plane of splice to which resistance of area A_n is tributary, mm.
X_u	ultimate strength of weld metal as rated by Electrode Classification number, MPa.
Y	ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel.
Y'_b	distance from centroid of the steel section under tension to the bottom fibre of the steel beam or girder, mm.
Y_t	distance from centroid of the steel beam to the top fibre of the beam, mm.
Y'_t	distance from centroid of the steel section under compression to the top fibre of the steel beam or girder, mm.
Y_{tc}	distance from the neutral axis of the composite section to the top fibre of the steel beam, mm.
Z	plastic section modulus of a steel section, mm^3.
β	coefficient used in moment resistance calculations for hybrid girders; or coefficient in the shear, tension interaction equation for bolts.
η	coefficient used in shear resistance calculations.
θ	angle of inclination of web plate of box girders, degrees.
λ	non-dimensional slenderness ratio in column formula.
μ	coefficient related to the slip resistance of a bolted joint.
ξ	coefficient used in determining moment of inertia of compression flange stiffeners.
ρ	specified minimum yield stress of the web divided by the specified minimum yield stress of the tension flange.
σ_1, σ_2	coefficients used in determining moment resistances of beams and girders.
τ	coefficient used in shear resistance calculations.
ϕ	performance factor for steel.
ϕ_b	performance factor for bolts.
ϕ_c	performance factor for concrete.
ϕ_r	performance factor for reinforcement.
ψ	coefficient used in moment resistance calculations for hybrid girders.
ω	coefficient used to determine equivalent uniform bending effect in beam columns.

Materials 10—4

General 10—4.1

10—4.1.1 Acceptable material and product standards and specifications shall conform to Clauses 10—4.2 to 10—4.6, inclusive, as applicable, or shall be subject to approval.

10—4.1.2 Steel grades specified for bridge members shall have low temperature notch toughness characteristics consistent with the conditions described in Section 11.

10—4.1.3 Unless approved, plates provided from coils shall not be used.

Structural Steel 10—4.2

Structural steel shall conform to the requirements of CAN3-G40.21-M81, *Structural Quality Steels*. The modulus of elasticity of structural steel, E, shall be taken as 200 000 MPa.

Cast Steel **10—4.3**

Cast steel shall conform to the requirements of either:

- CSA Standard G28, *Carbon-Steel Castings for General Application*; or
- ASTM Standard A486, *Steel Castings for Highway Bridges*.

Bolts **10—4.4**

10—4.4.1 Bolts shall conform to the requirements of either:

- ASTM Standard A325 or ASTM Standard A325M, *High-Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers*; or
- ASTM Standard A490 or ASTM Standard A490M, *Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints*.

10—4.4.2 Galvanized ASTM A490 bolts shall not be used. When uniform appearance is an important criterion, ASTM A490 bolts shall not be used with atmospheric corrosion-resisting steel.

Welding Electrodes **10—4.5**

Welding electrodes shall conform to the requirements of one of:

- CSA Standard W48.1, *Mild Steel Covered Arc-Welding Electrodes*, basic electrodes only;
- CSA Standard W48.3, *Low-Alloy Steel Arc-Welding Electrodes*, basic electrodes only;
- CSA Standard W48.4, *Solid Mild Steel Electrodes for Gas Metal Arc Welding*;
- CSA Standard W48.5, *Mild Steel Electrodes for Flux-Cored Arc Welding*, CH designation electrodes only;
- CSA Standard W48.6, *Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding*.

Stud Shear Connectors **10—4.6**

10—4.6.1 Stud shear connectors shall conform to the requirements of ASTM Standard A108, *Steel Bars, Carbon, Cold-Finished, Standard Quality, Grades 1015, 1018 or 1020*, either semi-killed or fully-killed.

10—4.6.2 Mechanical properties, as determined by tests conducted in accordance with the requirements of ASTM Standard A370, *Standard Methods and Definitions for Mechanical Testing of Steel Products*, of bar stock after drawing, or full diameter finished studs, shall conform to the following:

• minimum tensile strength	—	410 MPa
• minimum yield strength as determined by 0.2% offset method	—	350 MPa
• minimum elongation in 50 mm	—	20%
• minimum reduction of area	—	50%

Identification **10—4.7**

10—4.7.1 The materials and products used shall be identified as to specification, including type or grade if applicable, by:

a) Mill Test Certificates or producer's certificates satisfactorily correlated to the materials or products to which they pertain; or

b) legible markings on the material or product made by the producer in accordance with the applicable material or product standard.

Otherwise, the requirements of Clause 10—4.7.2 shall apply.

10—4.7.2 Structural steel not satisfactorily identified, as stipulated in Clause 10—4.7.1(a) or (b), shall not be used unless tested by an approved testing laboratory in accordance with the requirements of CAN3-G40.20-M81, *General Requirements for Rolled or Welded Structural Quality Steel*. The results of such testing, taking into account both mechanical and chemical properties, shall form the basis for classifying the steel.

Design Theory and Assumptions 10—5

Requirements at Serviceability Limit State Type I **10—5.1**

Structural members and components shall be proportioned for the effects of fatigue in accordance with the provisions of Section 11.

Requirements at Serviceability Limit State Type II **10—5.2**

Compact sections shall be proportioned such that yielding in the extreme fibres of the steel section under serviceability limit state Type II does not occur.

Bolted joints, in which the loads are assumed to be transferred by the slip resistance of the clamped faying surfaces, shall be proportioned in accordance with Clause 10—16.2.2.

Requirements at Ultimate Limit States **10—5.3**

The structural steelwork for bridges shall be proportioned to resist the load effects resulting from the application of the loads specified in Section 2 and taking into account the performance factors as specified in this section.

Fracture Control **10—5.4**

Fracture control requirements shall be in accordance with the provisions of Section 11.

Performance Factors **10—5.5**

The performance factor, ϕ, for steel shall be taken as 0.90.

In computing factored flexural resistance of shapes fabricated from plates, the performance factor, ϕ, shall be taken as 0.95.

The performance factor, ϕ_r, for reinforcing steel shall be taken as 0.85.

The performance factor, ϕ_c, for concrete when used in composite construction shall be taken as 0.70.

The performance factor, ϕ_b, for bolts shall be taken as 0.67.

Analysis **10—5.6**

10—5.6.1 **General**

Unless other methods are approved, the methods of analysis given in Section 3 and in this section as applicable shall be used.

The design of supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss is connected or supported.

Design Lengths of Members **10—5.7**

10—5.7.1 **Simple Span Flexural Components**

The span length of simple span beams and girders shall be taken as the distance between centres of bearings or the actual length measured centre-to-centre of end connections. The length of trusses designed as simple spans shall be taken as the distance between centres of bearings or the distance between the extreme working points of the system of triangulation employed.

10—5.7.2 **Continuous Span Flexural Components**

The span lengths of continuous beams, girders, and trusses shall be taken as the distance between adjacent points of support.

10—5.7.3 **Compression Members**

10—5.7.3.1 Compression members shall be designed on the basis of their effective length KL. The unbraced length, L, shall be taken as the length of the compression member measured centre-to-centre of restraints. The unbraced length may differ for buckling considerations in different planes. For the bottom level of a multi-level bent or for a single-level bent, L shall be measured from the top of the base plate.

10—5.7.3.2 Unless the degree of rotational restraint at the end of the unbraced lengths shows that a smaller value is applicable, the value of K shall be taken as 1.0 for compression members of frames in which:

a) the sway effects, in addition to the lateral loads, are resisted by bracing; or

b) sway effects have been included in the analysis.

10—5.7.3.3 For rigid bents in which sway effects have not been included in the analysis used to determine the moments and forces, the value of K shall be determined from the degree of rotational and translational restraint provided at the ends of the unbraced length, but shall be not less than 1.0.

10—5.7.3.4 **Compression Members in Trusses**

Unless otherwise specified in this section or unless analysis shows that a smaller value is applicable, the value of K shall be taken as 1.0 for compression members in trusses.

For half-through or pony-truss spans, the critical buckling load of the compression chord shall be determined in accordance with Clause 10—14.3.6.

Design Detail Requirements 10—6

General **10—6.1**

Unless otherwise specified in this section, the requirements of Clause 10—6 shall be met. Members and connections shall be detailed to minimize their susceptibility to fatigue and brittle fracture.

Corrosion Protection **10—6.2**

Corrosion protection shall be provided by means of suitable alloying elements in the steel, by protective coatings, or by other approved means. The type and degree of corrosion protection to be provided shall be shown on the plans.

Minimum Thickness of Material **10—6.3**

10—6.3.1 The minimum thickness of steel shall be as follows:

a) gusset plates for main members, splice plates, bracings, cross frames and all material in end floorbeams and end diaphragms and their connections, 10 mm;

b) closed members, such as tubular members or closed ribs in orthotropic decks, sealed against entry of moisture, 5 mm;

c) webs of rolled shapes, 6 mm; webs of plate girders and box girders, 10 mm;
d) other structural steel, except for fillers, railings and non-stressed items, 8 mm.

Accessibility and Drainage **10—6.4**

10—6.4.1 The accessibility of all parts of a structure where required for inspection, cleaning and painting, shall be ensured by the proper proportioning of members and the design of their details.

10—6.4.2 Pockets and depressions which retain water shall be avoided. Pockets shall be provided with effective drain holes or be filled with waterproofing material.

Floorbeams and Diaphragms at Supports **10—6.5**

10—6.5.1 Floorbeams and diaphragms at piers and abutments shall be designed to permit jacking of the superstructure under dead load.

10—6.5.2 Unless embedded in concrete or otherwise protected, end floorbeams and end diaphragms shall be so arranged as to permit painting of the side of the beam or diaphragm adjacent to the ballast wall of the abutment.

Camber **10—6.6**

10—6.6.1 Camber shall be shown on the plans. Where provided, it shall compensate for dead load deflections and in addition shall conform to the profile grade line when the structure is on a vertical curve.

10—6.6.2 Camber, amounting to not less than the percentages of the computed unfactored dead load deflection shown in Table 10—6.6.2, shall be provided.

For spans less than 25 m, the thickness of the concrete deck slab over longitudinal beams may be increased by haunching in lieu of providing a camber.

Table **Dead Load Camber Percentage** **10—6.6.2**

Span, m	Percentage, %
15-30	120
30-75	115
over 75	110

Anchorage Devices **10—6.7**

10—6.7.1 Anchorage devices provided to connect the superstructure to the substructure shall be adequate to withstand the effect of horizontal and vertical loads at ultimate limit states determined in accordance with Section 2.

10—6.7.2 Anchor bolts shall have a minimum diameter of 30 mm and a minimum embedment of 400 mm.

Welded Structures **10—6.8** Primary members of welded structures shall be of type W, type A, type Q, type WT, type AT, or type QT steels as specified in CAN3-G40.21-M81, *Structural Quality Steels*.

Welded Attachments **10—6.9** All attachments to primary members including transverse and longitudinal stiffeners shall be connected by continuous welds. Splicing of longitudinal stiffeners shall be by means of complete joint penetration groove welds.

Tension Members 10—7

General **10—7.1** Tension members shall be proportioned on the basis of the net cross-sectional area.

10—7.1.1 Net area shall be computed by summing the products of the thickness and the net width of each element taken normal to the axis of the member.

10—7.1.2 The net width of a member at a section through any series of holes across the member shall be determined by subtracting the total width of all holes along the section from the gross width of the section and adding for each gauge distance, g, in the series, the quantity $s^2/4g$.

The hole dimension used in the computation of the net width shall be taken as 2 mm larger than the specified width of the holes taken normal to the axis of the member.

10—7.1.3 The critical net area, A_n, shall be used to compute the tensile resistance; however, areas exceeding the following limits shall not be used in the computation:

- $0.85\ A_g$ when $F_y/F_u \leq 0.75$
- $0.90\ A_g$ when $0.75 < F_y/F_u \leq 0.85$
- $0.95\ A_g$ when $0.85 < F_y/F_u$

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10 — 7.1.4 All elements of rolled shapes and built-up cross-sections which are deemed to contribute to the net area shall be connected directly by the proportionate number of bolts or length of weld in the connection. Alternatively, in a lapped joint, an element which is not connected directly may be included in the net area computation if the total net area of the member is reduced according to the following relationship.

$$A_n' = \Sigma A_n \left(1 - \frac{\bar{x}}{L}\right)$$

10 — 7.1.5 The slenderness ratio of a tension member shall not exceed 200, unless otherwise approved.

Axial Tension

10 — 7.2 The factored resistance, T_r, of a member subjected to an axial tensile force, shall be taken as the lesser of:

a) $T_r = \phi A_n F_y$ when $A_n/A_g \geq F_y/F_u$, and

$T_r = \phi\left(F_u \dfrac{A_n}{A_g}\right)A_n$ when $A_n/A_g < F_y/F_u$, or

b) $T_r = 0.85\, \phi A_n F_u$

Axial Tension and Bending

10 — 7.3 Members subjected to moments and axial tensile force shall satisfy the following relationship.

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where: $M_r = \phi M_p$ for compact sections

$M_r = \phi M_y$ for non-compact sections

and $\dfrac{M_f}{M_r} - \dfrac{T_f Z}{M_r A} \leq 1.0$ for compact sections

or $\dfrac{M_f}{M_r} - \dfrac{T_f S}{M_r A} \leq 1.0$ for non-compact sections

where M_r is defined in Clauses 10 — 9.2 and 10 — 9.3, as appropriate.

Compression Members

10 — 8

General

10 — 8.1

10 — 8.1.1 Compression members shall be proportioned on the basis of the gross area of the cross-section computed by summing the products of the thickness and gross width of each element as taken normal to the axis of the member.

10 — 8.1.2 Provided that the requirements of Table 10 — 8.2.1 are met, the expressions for compressive resistance in this clause shall apply to hot-rolled W-shapes, fabricated I-shapes, welded H-shapes having flame-cut flange edges, fabricated box shapes, and cold-formed stress-relieved hollow structural sections.

The compressive resistance of asymmetric, singly symmetric, and cruciform sections shall be computed taking into account torsional-flexural buckling. The computation of compressive resistance of members, other than those covered specifically herein, shall be subject to approval.

10 — 8.1.3 The slenderness ratio shall not exceed 120 for main compression members nor 160 for secondary and bracing members.

Width-Thickness Ratio of Compression Elements

10 — 8.2

10 — 8.2.1 Classification of Sections

Rolled or fabricated sections having, when subject to flexure, an axis of symmetry in the plane of loading and meeting the width-to-thickness requirements for compact sections in Table 10 — 8.2.1 shall be classified as compact sections. Other sections shall be classified as non-compact sections and shall meet the width-to-thickness ratios for non-compact sections in Table 10 — 8.2.1.

10 — 8.2.2 For elements supported along one edge only, parallel to the direction of compressive loading, the width shall be taken as follows:

a) for plates, the width, b, is the distance from the free edge to the first line of bolts or welds;

b) for legs of angles, flanges of channels and tees, and stems of tees, the width, b, is the full nominal dimension;

c) for flanges of beams and tees, the width, b, is half of the full flange width.

Table **Width-Thickness Limitations** **10—8.2.1**

Description of Element	Section Classification Compact	 Non-Compact
Legs of angles and elements supported along one edge, except as noted.	—	$b/t \le 200/\sqrt{F_y}$
Angles in continuous contact with other elements; plate girder stiffeners	—	$b/t \le 260/\sqrt{F_y}$
Stems of T-sections	$b/t \le 170/\sqrt{F_y}$	$b/t \le 340/\sqrt{F_y}$
Flanges of I or T-sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact.	$b/t \le 170/\sqrt{F_y}$	$b/t \le 260/\sqrt{F_y}$
Flanges of channels.	—	$b/t \le 260/\sqrt{F_y}$
Flanges of rectangular hollow structural sections	$b/t \le 525/\sqrt{F_y}$	$b/t \le 670/\sqrt{F_y}$
Flanges of box sections, flange cover plates and diaphragm plates, between lines of fasteners or welds.	$b/t \le 525/\sqrt{F_y}$	$b/t \le 670/\sqrt{F_y}$
Perforated cover plates	—	$b/t \le 840/\sqrt{F_y}$
Webs in axial compression	$h/w \le 670/\sqrt{F_y}$	$h/w \le 670/\sqrt{F_y}$
Webs in flexural compression	$h/w \le 1370/\sqrt{F_y}$	$h/w \le 1810/\sqrt{F_y}$
Webs in combined flexural and axial compression	when: $C_f/C_y \le 0.15$, $h/w \le (1370/\sqrt{F_y})(1-1.28C_f/C_y)^*$ when: $C_f/C_y > 0.15$, $h/w \le (1180/\sqrt{F_y})(1-0.43C_f/C_y)^*$	when: $C_f/C_y \le 0.15$, $h/w \le (1810/\sqrt{F_y})(1-1.69C_f/C_y)^*$ $h/w \le (1470/\sqrt{F_y})(1-0.54C_f/C_y)^*$
Circular hollow sections.	$D/t \le 18000/\sqrt{F/_y}$	$D/t \le 23000/\sqrt{F/_y}$

*h/w need not be less than $670/\sqrt{F_y}$

10—8.2.3 For elements supported along two edges parallel to the direction of the compressive loading, the width shall be taken as follows.

a) For flange or diaphragm plates in built-up sections, the width, b, is the distance between adjacent lines of bolts or lines of welds.

b) For sides of rectangular hollow structural sections, the width, h, shall be taken as the clear distance between edge-supporting elements less the fillet or corner radius at each junction of the edge-supporting elements. The radius of the corner shall be taken as equal to the thickness of the element for hollow structural sections.

c) For webs of built-up sections, the width, h, is the distance between nearest lines of bolts connecting either edge of the web or the clear distance between flanges when welds are used.

d) For webs of rolled sections, the width, h, is the clear distance between flanges.

10—8.2.4 The thickness of elements shall be taken as the nominal thickness. For tapered flanges the thickness shall be taken as the thickness at the midpoint of the element under consideration.

Axial Compressive Resistance

10—8.3 The factored compressive resistance, C_r, of a member subjected to an axial compressive force and conforming to the limitations of Clauses 10—8.1 and 10—8.2, shall be taken as:

a) $0 \leq \lambda \leq 1.0;\ C_r = \phi A F_y(\ 1.035 - 0.202\lambda - 0.222\lambda^2)$

b) $1.0 < \lambda \leq 2.0;\ C_r = \phi A F_y(-0.111 + 0.636\lambda^{-1} + 0.087\lambda^{-2})$

c) $2.0 < \lambda \leq 3.0;\ C_r = \phi A F_y(\ 0.009 + 0.877\lambda^{-2})$

where: $\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}}$

Axial Compression and Bending

10—8.4

10—8.4.1 **General**

Members required to resist bending moments and axial compression shall be proportioned to meet the requirements of this clause and Clause 10—9.

10—8.4.1.1 **Compact Sections, H-Shapes**

Compact H-shape sections in accordance with Clause 10—9.2, shall satisfy the following relationships:

a) $\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$

where M_{rx} and M_{ry} are as given by M_r in Clause 10—9.2.3.

b) $\frac{C_f}{C_r} + 0.85\frac{M_{fx}}{M_{rx}} + 0.60\frac{M_{fy}}{M_{ry}} \leq 1.0$

where M_{rx} and M_{ry} are as given by M_r in Clause 10—9.2.3 and C_r is equal to $\phi A F_y$.

c) $\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left(1 - \frac{C_f}{C_{ex}}\right)} + \frac{\omega_y M_{fy}}{M_{ry}\left(1 - \frac{C_f}{C_{ey}}\right)} \leq 1.0$

where M_{rx} is as given in Clause 10—9.2.4, M_{ry} is as given in Clause 10—9.2.3, ω is as given in Clause 10—8.4.2, and C_r is as given in Clause 10—8.3 and C_{ex} and C_{ey} are Euler buckling forces with respect to X-X and Y-Y axes, respectively.

10—8.4.1.2 **Other Singly- and Doubly-Symmetric Sections**

Other singly- and doubly-symmetric sections shall satisfy the following relationships:

a) $\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$

where M_r is as given in Clause 10—9.3.2 and C_r is equal to $\phi A F_y$;

b) $\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\left(1 - \frac{C_f}{C_{ex}}\right)} + \frac{\omega_y M_{fy}}{M_{ry}\left(1 - \frac{C_f}{C_{ey}}\right)} \leq 1.0$

where M_{rx} is as given in Clause 10—9.3.3, M_{ry} is as given in Clause 10—9.3.2, ω is as given in Clause 10—8.4.2, and C_r is as given in Clause 10—8.3.

10—8.4.2 **Moment Gradient Effects**

In the absence of a detailed analysis of the effects of moment gradients, the following values shall be used for ω.

a) Members not subjected to transverse loads between supports:

When $K \leq 1.0$,

$\omega = 0.6 + 0.4\, M_{f1}/M_{f2}$ for members bent in single curvature,

$\omega = 0.6 - 0.4\, M_{f1}/M_{f2}$ but not less than 0.4 for members bent in double curvature,

where M_{f1}/M_{f2} = ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, in the plane of bending considered.

When $K > 1.0$,

$\omega = 0.85$ for members bent in double curvature or subject to moment at only one end,

ω = 1.0 for members bent in single curvature due to moment at both ends.

b) Members subjected to distributed load or more than one concentrated load between supports: $\omega = 1.0$.

c) Members subjected to concentrated load or moment between supports: $\omega = 0.85$.

Beams and Girders 10—9

General 10—9.1

10—9.1.1 Beams and girders shall be proportioned on the basis of the geometric properties of the gross section or the modified gross section. No deduction shall be made for bolt holes in flanges unless the reduction of flange area by such holes exceeds 15% of the gross flange area, in which case the amount removed in excess of 15% shall be deducted from the gross flange area.

10—9.1.2 Flanges of welded beams or girders shall consist of single plates or a series of plates, joined end-to-end by complete penetration groove welds. Partial length cover plates shall not be used unless approved.

10—9.1.3 Lateral support of compression flanges shall be provided by secure connection to the concrete slab or to a steel floor deck, or by braces capable of restricting lateral displacement and twisting of the beams and girders. Timber floors shall not be considered to provide adequate lateral support unless the floor and fastenings are designed for this purpose.

10—9.1.4 Welds connecting flanges to webs shall be proportioned to resist interface shear due to bending combined with any loads which may be transmitted from the flange to the web other than by direct bearing.

10—9.1.5 When webs are subjected to the simultaneous action of shear and flexure, and when $V_f/V_r > 0.6$, the following flexure-shear interaction formula shall be satisfied:

$$0.727\,M_f/M_r + 0.455\,V_f/V_r < 1.0.$$

Compact Sections 10—9.2

10—9.2.1 Rolled or fabricated beams and girders subject to flexure, having an axis of symmetry in the plane of loading and meeting the width-to-thickness requirements of Clause 10—9.2.2 shall be classified as compact sections.

10—9.2.2 Compact sections shall meet the following requirements:

a) for projecting compression flange elements, $b/t \leq 170/\sqrt{F_y}$; and

b) for the web, $h/w \leq 1370/\sqrt{F_y}$.

10—9.2.3 For doubly-symmetric compact sections, the factored moment resistance shall be computed as:

$$M_r = \phi Z F_y = \phi M_p$$

provided that the compression flange is continuously supported in the lateral direction or that lateral bracing is provided for the compression flange in accordance with the following requirements:

$$L \leq 550 r_y/\sqrt{F_y} \text{ for } M_1/M_2 > 0.5$$

or

$$L \leq 980 r_y/\sqrt{F_y} \text{ for } M_1/M_2 \leq 0.5$$

where M_1/M_2 = the ratio of the smaller moment to the larger moment at opposite ends of the unbraced length, at ultimate limit state, in the plane of bending considered.

10—9.2.4 For doubly-symmetric compact sections which are bent about their major axis, and which do not meet the lateral support or lateral bracing requirements of Clause 10—9.2.3, the factored moment resistance shall be computed as:

a) when $M_u > 2/3\,M_p$,

$$M_r = 1.15\,\phi\,M_p\left[1 - \frac{0.28\,M_p}{M_u}\right] \leq \phi\,M_p$$

b) when $M_u \leq 2/3\,M_p$,

$$M_r = \phi\,M_u$$

where: $M_u = \frac{\pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$

For I-shaped members bent about the major axis, M_u may be taken as:

$$M_u = S\sqrt{\sigma_1^2 + \sigma_2^2}$$

where: $\sigma_1 = 140\,000/(Ld/A_{cf})$, and

$\sigma_2 = 1\,700\,000/(L/r_t)^2$

For compact sections which are not doubly symmetric, an approved method of analysis shall be used.

10 — 9.2.5 Transversely-stiffened compact sections, where $h/w > 502\sqrt{k_v/F_y}$, shall meet the following requirements:

$V_f/V_r \leq 1.0$, and

$M_f/M_r \leq 1.0$, and

$0.727\,M_f/M_r + 0.455\,V_f/V_r \leq 1.0$.

Non-Compact Sections

10 — 9.3

10 — 9.3.1 Non-compact doubly-symmetric sections shall meet the following requirements:

a) for projecting compression flange elements:

$b/t \leq 260/\sqrt{F_y}$, and

b) for the web:

$h/w \leq 1810/\sqrt{F_y}$

10 — 9.3.2 For non-compact doubly-symmetric sections, the factored moment resistance shall be computed as:

$$M_r = \phi S F_y = \phi M_y$$

provided that the compression flange is continuously supported in the lateral direction or that lateral bracing is provided for the compression flange in accordance with the following requirements:

$$L \leq \frac{140\,000\,A_f}{F_y d} \text{ or } L \leq \sqrt{\frac{1\,700\,000\,r_t^2}{F_y}}\text{, whichever is greater.}$$

10 — 9.3.3 For non-compact doubly-symmetric sections not meeting the lateral support or lateral bracing requirements of Clause 10 — 9.3.2, the factored moment resistance shall be computed as:

a) when $M_u > 2/3\,M_y$,

$$M_r = 1.15\,\phi M_y\left[1 - \frac{0.28\,M_y}{M_u}\right] \leq \phi M_y$$

b) when $M_u \leq 2/3\,M_y$,

$$M_r = \phi M_u$$

where: $M_u = \frac{\pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_yC_w}$

For I-shaped members bent about the major axis, M_u may be taken as:

$$M_u = S\sqrt{\sigma_1^2 + \sigma_2^2}$$

where: $\sigma_1 = 140\,000/(Ld/A_{cf})$, and

$\sigma_2 = 1\,700\,000/(L/r_t)^2$

For non-compact sections which are not doubly symmetric, an approved method of analysis shall be used.

10 — 9.3.4 Transversely-stiffened non-compact sections with $h/w > 502\sqrt{k_v/F_y}$ shall meet the requirements of Clause 10 — 9.2.5.

Sections with Stiffened Webs

10 — 9.4

10 — 9.4.1 Stiffened plate girders shall meet the following requirements:

a) for projecting compression flange elements:

$$b/t \leq 260/\sqrt{F_y}, \text{ and}$$

b) for the web:

$$h/w \leq 3150/\sqrt{F_y}$$

except as modified in Clause 10—9.8.

10—9.4.2 When $h/w > 3150/\sqrt{F_y}$, longitudinal stiffeners shall be provided in accordance with Clause 10—9.8.

10—9.4.3 For stiffened plate girders the moment resistance shall be determined in accordance with Clause 10—9.3.2 or Clause 10—9.3.3, as applicable.

When $h/w > 2550/\sqrt{F_y}$, moment resistance shall be reduced by a factor

$$\left[1.0 - 0.0005\,\frac{A_w}{A_{cf}}\left(\frac{h}{w} - \frac{2550}{F_y}\right)\right]$$

When $h/w > 2550/\sqrt{F_y}$ and $A_w/A_{cf} > 2$, an approved method of analysis shall be used.

Shear Resistance

10—9.5 The factored shear resistance for compact sections, non-compact sections, and sections with stiffened webs shall be computed as:

$$V_r = \phi A_w F_s$$

where A_w, the shear area, is computed using d_b for rolled shapes and h for girders, and F_s, the ultimate shear stress, MPa, is as follows:

a) for $\frac{h}{w} \leq 502\sqrt{\frac{k_v}{F_y}}$ $\qquad F_s = \frac{290\sqrt{F_y k_v}}{(h/w)} \leq 0.58\,F_y$

b) for $502\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 621\sqrt{\frac{k_v}{F_y}}$ $\qquad F_s = \left[\frac{290\sqrt{F_y k_v}}{(h/w)}\right]\tau + \eta F_y$

c) for $\frac{h}{w} > 621\sqrt{\frac{k_v}{F_y}}$ $\qquad F_s = \left[\frac{180\,000\,k_v}{(h/w)^2}\right]\tau + \eta F_y$

where: $k_v = 4 + \frac{5.34}{(a/h)^2}$ when $a/h < 1$, or

$k_v = 5.34 + \frac{4}{(a/h)^2}$ when $a/h \geq 1$, and

$\tau = 1 - \frac{0.866}{\sqrt{1 + (a/h)^2}}$, and

$\eta = \frac{0.50}{\sqrt{1 + (a/h)^2}}$

For unstiffened webs, $a/h = \infty$, $\tau = 1$, $\eta = 0$ and $k_v = 5.34$

Bearing Stiffeners

10—9.6

10—9.6.1 Bearing stiffeners shall be provided where the concentrated loads or reactions at ultimate limit states exceed the factored compressive resistances of webs of rolled beams and welded plate girders at the web toe of the flange-to-web fillets computed as follows:

a) for interior loads:

$$B_r = 1.25\,\phi\,w(N + 2k)\,F_y, \text{ and}$$

b) for end reactions:

$$B_r = 1.25\,\phi\,w(N + k)\,F_y$$

10—9.6.2 **Bearing stiffeners shall bear against the flanges through which the loads are transmitted and shall extend as close to the edge of the flanges as practicable. They shall be designed as compression members in accordance with Clause 10—8, assuming a column section to consist of a pair of stiffeners and a centrally-located strip of the web equal to 20 times its thickness at interior stiffeners, and a pair of stiffeners and a strip equal to 10 times its thickness when stiffeners are located at the end of the web.**

The effective column length shall be taken as not less than 0.75 of the length of the stiffeners. Only that portion of the stiffeners outside the fillet or the flange-to-web welds of the beam or girder shall be considered effective in bearing. Connections shall be designed for the interface force transmitted from the web to the stiffener.

Intermediate Transverse Stiffeners

10—9.7

10—9.7.1 Intermediate transverse stiffeners, when used, shall be spaced to provide shear resistance as determined from the formula given in Clause 10—9.5 except that at end panels the smaller dimension, a or h, shall not exceed $1160\, w/\sqrt{V_f/\phi A_w}$ where V_f is the largest shear force in the panel at ultimate limit state.

10—9.7.2 The maximum distance between stiffeners shall not exceed 2h. Intermediate transverse stiffeners may be provided on one or both sides of the web. The moment of inertia of the stiffener or stiffeners if so furnished, shall not be less than aw^3j mm^4 taken about the axis in the plane of the web, where $j = 2.5(h/a)^2 - 2$, but not less than 0.5. When stiffeners are provided on one side of the web only, the moment of inertia shall be taken at the face in contact with the web. The gross area of an intermediate stiffener or stiffeners shall not be less than

$$A_s = \frac{aw}{2}\left[1 - \frac{a/h}{1 + (a/h)^2}\right]CYD$$

where: $C = 1 - \frac{310\,000\, k_v}{F_y\,(h/w)^2}$ but not less than 0.10,

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel,

D = 1.0 for stiffeners provided in pairs,

D = 1.8 for single angle stiffeners, and

D = 2.4 for single plate stiffeners.

When the maximum shear force at ultimate limit state, V_f, in an adjacent panel is less than that permitted by Clause 10—9.5, this gross area requirement may be reduced by multiplying by the ratio V_f/V_r.

The width-to-thickness ratio of intermediate transverse stiffeners shall not exceed $260/\sqrt{F_y}$.

10—9.7.3 The connection between the web and stiffener or stiffeners shall be designed for a shear force of $1 \times 10^{-4} h\, F_y^{1.5}$ N/mm of web depth, h. When the largest computed shear force at ultimate limit state, V_f, in the adjacent panels is less than V_r computed by Clause 10—9.5 this shear force requirement may be reduced in the proportion of V_f/V_r. However, the total shear force shall in no case be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener.

10—9.7.4 When intermediate stiffeners are used on only one side of the web or when lateral bracing is attached to the stiffeners, they shall be connected to the compression flange of the girder. This connection shall be designed for a force of at least 2% of the total flange force at ultimate limit state at that point. Other intermediate stiffeners shall have at least a snug fit against the compression flange and if the stiffener is cut short of the tension flange, the distance between the end of the stiffener weld and the near edge of the web-to-flange fillet weld shall be between 4w and 6w. The stiffeners shall be clipped so as to clear the flange-to-web welds on welded girders. The width of the plate stiffener shall be not less than 50 mm plus 1/30 the depth of the girder, and shall not be less than 1/4 the full width of the flange.

Longitudinal Stiffeners

10—9.8

10—9.8.1 When one longitudinal stiffener is used it shall be placed at a distance 0.2h from the inner surface of the compression flange. If more than one longitudinal stiffener is used, the design shall be based on approved methods of analysis.

The web slenderness ratio of plate girders with transverse stiffeners and one longitudinal stiffener shall meet the requirement:

$$h/w \leq 6000/\sqrt{F_y}.$$

Longitudinal stiffeners and transverse stiffeners shall be placed on opposite sides of the girder web, unless otherwise approved.

10—9.8.2 The longitudinal stiffener shall be proportioned so that:

a) $b/t \leq 200/\sqrt{F_y}$,

b) $I \geq hw^3 [2.4 (a/h)^2 - 0.13]$, and

c) $r \geq a\sqrt{F_y}/1900$.

I and r shall be computed for a section comprising a stiffener or stiffeners and a centrally-located web strip equal to 20w in width, with respect to the centroidal axis of this section, parallel to the web. Transverse stiffeners for girder panels with longitudinal stiffeners shall meet the requirements of Clause 10—9.7, except that the slenderness ratio of the transverse stiffener, b/t, shall not exceed $200/\sqrt{F_y}$ and the greater depth of the subpanel, h_p, shall be used in place of h in applying Clauses 10—9.7.2 and 10—9.7.3. The section modulus of the transverse stiffener shall be not less than $S_t = hS_h/3a$.

Lateral Bracing, Cross Frames, and Diaphragms 10—9.9

10—9.9.1 Lateral bracing, cross frames, and diaphragms for beam and girder bridges shall be provided in accordance with the requirements of this clause, unless a method of analysis as specified in Section 3 or otherwise approved, indicates that other arrangements are adequate.

10—9.9.2 Beam and girder bridges shall be provided with cross frames or diaphragms at piers and abutments. These cross frames or diaphragms shall be proportioned to transmit all lateral forces to the bearings. Cross frames and diaphragms shall be as deep as practicable. The requirements of Clause 10—6.5.1 shall also be met.

10—9.9.3 Unless otherwise indicated by analysis or construction requirements, intermediate cross frames or diaphragms shall be provided at intervals not exceeding 8 m.

10—9.9.4 Unless approved otherwise, beams and girders with spans greater than 50 m shall be provided with a system of lateral bracing located near the plane of bottom flanges. When a lateral bracing system is provided, its effect on the structural behaviour of the bridge shall be taken into account in the design, as specified in Section 3.

10—9.9.5 Bracing systems for the compression flanges of beams or girders shall meet the spacing requirements of Clauses 10—9.2.3, 10—9.3.2, or 10—9.4.2, as applicable. Bracing systems shall be designed at ultimate limit states in accordance with the provisions of Section 2, in addition to a lateral load equivalent to 1.0% of the compression flange force in the beam or girder at ultimate limit state.

Composite Beams and Girders 10—10

General 10—10.1

This clause pertains to structures composed of steel beams or girders and a concrete slab, in which resistance to shear at the interface between the slab and the beams or girders is provided by mechanical shear connectors.

Proportioning 10—10.2

10—10.2.1 The web of the steel section shall be designed to carry the total vertical shear and shall meet the requirements of Clause 10—9.

10—10.2.2 The modulus of elasticity of concrete shall be determined in accordance with Section 8.

10—10.2.3 To account for the effect of creep due to dead load, in lieu of more detailed computations, a modular ratio of 3n shall be used in computing the section properties. For unshored construction, this effect shall be determined for that portion of the dead load which is applied after the concrete strength has reached $0.75f'_c$. For shored construction, this effect shall be determined for all dead loads present.

Effective Slab Width 10—10.3

The effective slab width shall be determined in accordance with Section 3.

Beams During Construction 10—10.4

The steel section shall be proportioned to support all factored loads applied before the concrete strength reaches $0.75f'_c$.

Compact Sections 10—10.5

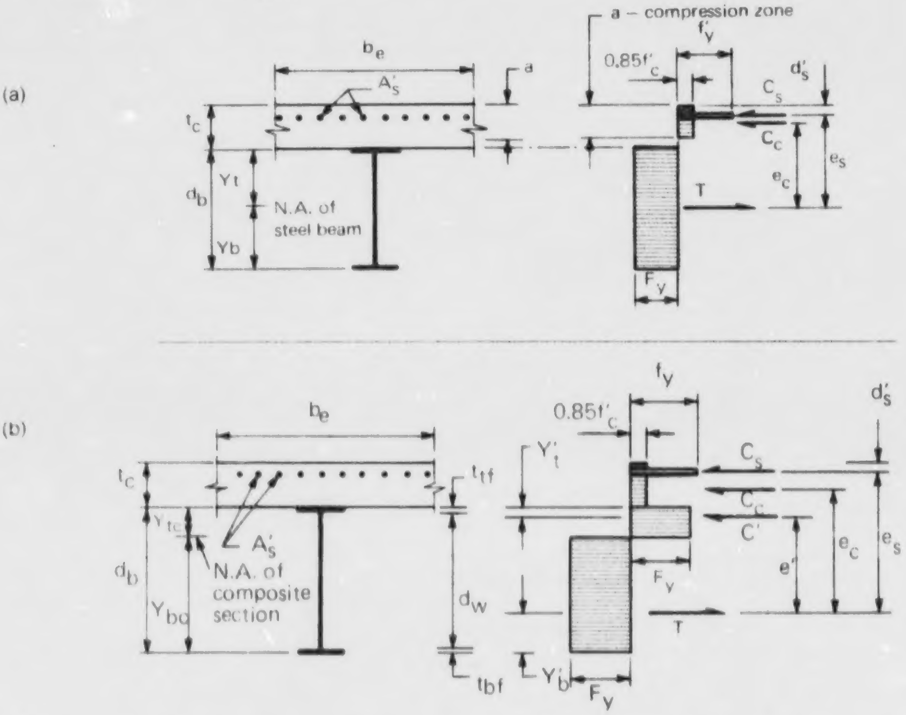
10—10.5.1 **General**

The steel section shall meet the requirements of Clause 10—9.2.

10—10.5.2 **Positive Moment Regions**

10—10.5.2.1 The factored moment resistance of the section in bending shall be computed using a fully-plastic stress distribution as shown in Figures 10—10.5.2.1(a) and 10—10.5.2.1(b).

Figure **10—10.5.2.1**


(a)
b_e
A'_s
a
t_c
Yt
d_b
N.A. of steel beam
Yb
a – compression zone
0.85f'_c
f'_y
d'_s
C_s
C_c
e_s
e_c
T
F_y
(b)
b_e
t_tf
t_c
Y_tc
A'_s
N.A. of composite section
d_b
Y_bc
d_w
t_bf
Y'_t
0.85f'_c
f_y
d'_s
C_s
C_c
C'
F_y
e'
e_c
e_s
T
Y'_b
F_y

10—10.5.2.2 The factored compressive resistance of the slab used in computing the factored resistance of the section shall be the smallest value computed from the following equations:

$$C_1 = C_c + C_s$$
$$C_2 = \phi A_s F_y$$
$$C_3 = \Sigma Q_r$$

where: performance factors are as given in Clause 10—5.5 and

$C_c = 0.85\,\phi_c f'_c b_e t_c$

$C_s = \phi_r A'_s f'_y$

ΣQ_r = sum of the factored resistances of shear connectors between the section under consideration and the section of zero moment, calculated in accordance with Clause 10—10.9.3.

10—10.5.2.3 When $C_1 > C_2$, the depth of the compressive stress block, a, shall be computed from:

$$a = \frac{C_2 - \phi_r A'_s f'_y}{0.85\,\phi_c b_e f'_c}$$

and the factored moment resistance of the section shall be computed from:

$$M_r = C_c e_c + C_s e_s$$

where: $C_c = 0.85\,\phi_c f'_c b_e a$

$e_c = Y_t + t_c - 0.5a$

$e_s = Y_t + t_c - d'_s$

10—10.5.2.4 When $C_1 < C_2$, the depth of the compressive stress block, a, shall be taken as equal to t_c and the factored moment resistance shall be computed from:

$$M_r = C_c e_c + C_s e_s + C'e'$$

where:

$$C_c = 0.85\,\phi f'_c b_e t_c$$

$$C' = 0.5\,(\phi A_s F_y - C_1)$$

$$e_c = d_b + 0.5\,t_c - Y'_b$$

$$e_s = d_b + t_c - d'_s - Y'_b$$

$$e' = d_b - Y'_b - Y'_t$$

$$Y_{tc} = \frac{C'}{\phi A_{tf} F_y}\, t_{tf} \qquad \text{when } C' < \phi A_{tf} F_y$$

or,

$$Y_{tc} = t_{tf} + \frac{C' - \phi A_{tf} F_y}{\phi A_w F_y}\, d_w \qquad \text{when } C' \geqslant \phi A_{tf} F_y$$

10—10.5.2.5 When the smallest factored compressive resistance computed in Clause 10—10.5.2.2 is C_3, the factored moment resistance shall be computed from:

$$M_r = \Sigma Q_r (d_b - Y'_b + t_c/2) + C'e'$$

10—10.5.3 **Negative Moment Regions**

10—10.5.3.1 When slab reinforcement is continuous over interior supports, or when the slab is prestressed longitudinally in accordance with Section 9 and when shear connectors are provided, the factored moment resistance of the section shall be based on a fully-plastic stress distribution using the appropriate performance factors for the steel and the reinforcement or prestressing strands used.

10—10.5.3.2 When shear connectors are not provided in the negative moment regions, the factored moment resistance shall be that of the steel section, computed in accordance with Clause 10—9.2.

10—10.5.3.3 The minimum longitudinal reinforcement in non-prestressed slabs, including longitudinal distribution reinforcement, shall be not less than 1% of the cross-section area of the slab.

At least two-thirds of this required reinforcement shall be placed in the top layer of slab reinforcement within the effective width of the slab.

10—10.5.3.4 When longitudinal reinforcement is considered a part of a composite section, the range of stress in slab reinforcement shall meet the fatigue requirements of Section 11.

10—10.5.3.5 When shear connectors are not provided in the negative moment region, the longitudinal reinforcement shall be extended into the positive moment regions in accordance with Section 8.

Non-Compact Sections

10—10.6

10—10.6.1 **Positive Moment Regions**

10—10.6.1.1 The factored moment resistance of composite sections in which the depth of the compressive portion of the web of steel section, D_c, does not exceed $685w/\sqrt{F_y}$, shall be determined in accordance with Clause 10—10.5.2.

10—10.6.1.2 When the steel section does not satisfy the requirements of Clause 10—10.6.1.1, but meets the requirements of Clause 10—9.3.1, the factored moment resistance of the composite section shall be taken either as the resultant moment based on the linear stress distribution at first yielding of the steel section, or by assuming a linear stress distribution in the steel section at first yielding of the steel section and a plastic stress distribution in the concrete slab.

In either case, the following condition shall be satisfied.

a) For shored beams:

$$\frac{M_d + M_{sdl}}{S_{3n}} + \frac{M_{ll}}{S_n} \leqslant \phi F_y$$

b) For unshored beams:

$$\frac{M_d}{S} + \frac{M_{sdl}}{S_{3n}} + \frac{M_{ll}}{S_n} \leqslant \phi F_y$$

10—10.6.2 **Negative Moment Regions**

10—10.6.2.1 When the steel section meets the requirements of Clause 10—9.3.1, the factored moment resistance shall be determined in accordance with Clause 10—9.3.2. The requirement of Clause 10—10.5.3.5 shall also be satisfied.

10—10.6.2.2 When shear connectors are provided and slab reinforcement is considered to act compositely with the steel section, and the requirements of Clauses 10—10.5.3.3 and 10—10.5.3.4 are met, the following condition shall be satisfied:

$$\frac{M_d}{S} + \frac{M_{sdl} + M_{ll}}{S'} \le \phi F_y$$

Sections with Stiffened Webs

10—10.7

10—10.7.1 **Positive Moment Regions**

When the steel section satisfies the requirements of Clause 10—9.4, the factored moment resistance shall be computed in accordance with Clause 10—10.6.1.2.

10—10.7.2 **Negative Moment Regions**

When the steel section satisfies requirements of Clause 10—9.4, the factored moment resistance shall be determined in accordance with Clause 10—9.3.2. The requirements of Clause 10—10.5.3.5 shall also be satisfied. When slab reinforcement is considered to act compositely with the steel section and the requirements of Clauses 10—10.5.3.3 and 10—10.5.3.4 are met, the factored moment resistance shall be computed using section comprised of steel section and reinforcement and the requirements of Clause 10—10.6.2.2 shall be satisfied.

Unsymmetrical Sections

10—10.8

10—10.8.1 **General**

For beams and girders symmetrical about the vertical axis but unsymmetrical about the horizontal centroidal axis, the provisions of Clause 10—9 shall apply, except that the depth of the compression portion of the web shall not exceed $685w/\sqrt{F_y}$ for compact sections and $905w/\sqrt{F_y}$ for non-compact sections.

10—10.8.2 **Transversely-Stiffened Webs**

Girders with transversely-stiffened webs shall be proportioned in accordance with the provisions of Clause 10—9, except that the depth of the compression portion of the web shall not exceed $1575w/\sqrt{F_y}$.

10—10.8.3 **Longitudinally-Stiffened Webs**

Longitudinal stiffeners shall be provided when the web thickness is less than that specified in Clause 10—9.4.

For girders with one longitudinal stiffener and transverse stiffeners, the provisions of Clause 10—9.8 shall apply, provided that:

a) the longitudinal stiffener is placed 0.4 D_c from the inner surface of the compression flange, and

b) the depth of the compression portion of the web does not exceed $3150w/\sqrt{F_y}$.

Shear Connectors

10—10.9

10—10.9.1 **General**

10—10.9.1.1 Shear connectors shall conform to the respective materials specification of Clause 10—4 and shall be capable of resisting both horizontal and vertical movement between the concrete slab and the steel.

10—10.9.1.2 Shear connectors shall also meet the fatigue requirements of Section 11.

10—10.9.1.3 Shear connectors shall be provided throughout simple spans in the positive moment regions of continuous spans. In negative moment regions, shear connectors shall be provided when slab reinforcement is considered in the computation of the section properties or when the slab is prestressed to achieve composite action.

When shear connectors are not provided in negative moment regions, additional connectors shall be provided as specified in Section 11.

10—10.9.2 **Placement, Spacing, Cover and Edge Distances**

10—10.9.2.1 Shear connectors shall be spaced uniformly or at variable intervals according to the variation in the interface shear.

Spacing of shear connectors shall not exceed 600 mm except to comply with fatigue requirements over the interior supports of continuous beams or girders in which case a wider spacing may be used.

10—10.9.2.2 The clear depth of concrete cover over the tops of shear connectors shall meet the requirements of Section 8. Shear connectors shall penetrate at least 25 mm above the plane of bottom slab reinforcement or shear reinforcement in the slab haunch.

10 — 10.9.2.3 The clear distance between the edge of a girder flange and a shear connector shank shall not be less than 25 mm.

10 — 10.9.3 **Number of Shear Connectors**

The number of shear connectors required at ultimate limit state shall be computed from:

$$N = \frac{P}{0.85\,Q_r}$$

where: $Q_r = 0.4\,d^2\sqrt{f'_c E_c}$ for shear connectors with a ratio of $H/d = 4.0$.

a) Between the points of maximum positive moment and points of adjacent end supports, P shall be taken as the smaller of:

$$P_1 = A_s F_y, \text{ and}$$

$$P_2 = 0.85\,f'_c b_c t_c + A'_s f'_y$$

b) Between the points of maximum positive moment and points of adjacent maximum negative moments, P shall be taken as the smaller of:

$$P_1 + P_3, \text{ and } P_2 + P_3,$$

where: $P_3 = A'_s f'_y$

Composite Box Girders

10 — 11

General

10 — 11.1 This clause pertains to the design of simple and continuous span bridges consisting of two or more single-cell composite box girders. It is applicable to box girders having a width centre-to-centre of top steel flanges in the same cell approximately equal to the distance centre-to-centre of adjacent top steel flanges of adjacent cells. The cantilever overhang of the deck slab, including curbs and parapets, shall be limited to 60% of the distance between the centres of adjacent top steel flanges of adjacent cells.

Particular attention shall be given to the possibility that tension flanges may be subjected to compressive forces during construction.

Moment Resistances

10 — 11.2

10 — 11.2.1 The factored moment resistance of box girders shall be determined in accordance with Clauses 10 — 9 and 10 — 10 except that contribution of the compression flange to the factored moment resistance in regions of negative moment shall be determined in accordance with Clauses 10 — 11.2.2, 10 — 11.2.3 and 10 — 11.2.4.

The effective width of bottom flange plates in tension shall be determined in accordance with Clause 10 — 11.4.

10 — 11.2.2 **Compression Flanges, Unstiffened**

The factored moment resistance shall be taken as:

a) when $b/t \leq 510/\sqrt{F_y}$,

$$M_r = \phi F_y S$$

b) when $510/\sqrt{F_y} < b/t \leq 1100/\sqrt{F_y}$,

$$M_r = \phi F_{cr} S$$

where $F_{cr} = 0.592\,F_y\left(1 + 0.687\sin\frac{\pi c}{2}\right)$

and $c = \dfrac{1100 - b/t\sqrt{F_y}}{590}$

c) when $b/t > 1100/\sqrt{F_y}$,

$$M_r = \phi F_{cr} S$$

where $F_{cr} = \dfrac{7.24 \times 10^5}{(b/t)^2}$

10 — 11.2.3 **Compression Flanges Stiffened Longitudinally**

The factored moment resistance shall be taken as:

a) when $b_s/t \leq \dfrac{255\sqrt{k_1}}{\sqrt{F_y}}$

$$M_r = \phi F_y S$$

---

b) when $\dfrac{255\sqrt{k_1}}{\sqrt{F_y}} < b_s/t \leq \dfrac{550\sqrt{k_1}}{\sqrt{F_y}}$

$$M_r = \phi F_{cr} S$$

where $F_{cr} = 0.592\,F_y\left(1 + 0.687\sin\dfrac{\pi c}{2}\right)$

and $c = \dfrac{550\sqrt{k_1} - (b_s/t\sqrt{F_y})}{295\sqrt{k_1}}$

---

c) when $b_s/t > \dfrac{550\sqrt{k_1}}{\sqrt{F_y}}$

$$M_r = \phi F_{cr} S$$

where $F_{cr} = \dfrac{18k_1 \times 10^4}{(b_s/t)^2}$

The longitudinal stiffeners shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to:

$$I_s = \xi t^3 b_s$$

where: $\xi = 0.07\,k_1^3 n^4$ for values of n greater than 1

$\xi = 0.125\,k_1^3$ for a value of $n = 1$

n = number of longitudinal stiffeners

k_1 = buckling coefficient which shall not exceed 4.0

At least one transverse stiffener shall be placed near the point of dead load contraflexure. This transverse stiffener shall have a size equal to that of a longitudinal stiffener.

10—11.2.4 Compression Flanges Stiffened Longitudinally and Transversely

The factored moment resistance shall be taken as:

---

a) when $b_s/t \leq \dfrac{255\sqrt{k_2}}{\sqrt{F_y}}$

$$M_r = \phi F_y S$$

---

b) when $\dfrac{255\sqrt{k_2}}{\sqrt{F_y}} < b_s/t \leq \dfrac{550\sqrt{k_2}}{\sqrt{F_y}}$

$$M_r = \phi F_{cr} S$$

where: $F_{cr} = 0.592\,F_y\left(1 + 0.687\sin\dfrac{\pi c}{2}\right)$

and $c = \dfrac{550\sqrt{k_2} - (b_s/t\sqrt{F_y})}{295\sqrt{k_2}}$

---

c) when $b_s/t > \dfrac{550\sqrt{k_2}}{\sqrt{F_y}}$

$$M_r = \phi F_{cr} S$$

where $F_{cr} = \dfrac{18k_2 \times 10^4}{(b_s/t)^2}$

$$k_2 = \frac{[1 + (a/b)^2]^2 + 87.3}{(n + 1)^2 (a/b)^2 [1 + 0.1(n+1)]}$$

The longitudinal stiffeners shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to:

$$I_s = 8t^3 b_s$$

The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge is at least equal to:

$$I_t = 0.055(n + 1)^3 b_s^3 \frac{F_{cr} A_f}{E\,a}$$

The ratio a/b shall be limited to a maximum value of 3.0. The maximum value of the buckling coefficient k_2 shall be 4.0. When k_2 has its maximum value, the transverse stiffeners shall have a spacing, a, equal to or less than $4b_s$. The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of the box and to each longitudinal stiffener. The connection to the web shall be proportioned for a vertical force equal to:

$$R_v = \frac{F_y S_t}{2b}$$

The connection to each longitudinal stiffener shall be proportioned for a vertical force equal to:

$$R_s = \frac{F_y S_t}{nb}$$

Compression Flange Stiffeners **10 — 11.3**

The width-to-thickness ratio of any outstanding element of the flange stiffeners shall not exceed the value:

$$\frac{b}{t} = \frac{200}{\sqrt{F_y}}$$

Effective Width of Tension Flanges **10 — 11.4**

The effective width of bottom-flange plates in tension shall be taken as not more than one-fifth of the span for simply-supported structures and not more than one-fifth of the distance between points of dead load contraflexure for continuous structures.

Web Plates **10 — 11.5**

Webs shall be proportioned in accordance with Clause 10—9. The shear force to be considered on each web shall be $V_f/\cos\theta$ where V_f = one-half of the total vertical shear force at the ultimate limit state on one box girder and θ = the angle of inclination of the web plate to the vertical. The inclination of the web plates shall not exceed 1 horizontal to 4 vertical.

Flange-to-Web Welds **10 — 11.6**

The total effective thickness of the flange-to-web welds shall be not less than the thickness of the web. If fillet welds are used, they shall be on both sides of the connecting flange or web plate.

Diaphragms **10 — 11.7**

Diaphragms, cross frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion. Flexural distortion and warping shall be investigated and diaphragms, cross frames, or other means shall be used to ensure that the resulting deformations of the boxes are adequately controlled during fabrication, transportation, erection, and placement of the deck, as well as under final loading conditions.

Diaphragms, cross frames, or other means shall be provided between the boxes at suitable intervals, so as to limit their relative movement under the action of unequal loading to a value consistent with the provisions of Section 8 regarding the strength and cracking limitations of slabs.

Hybrid Girders 10 — 12

General **10 — 12.1**

This clause provides design requirements for simple and continuous composite and non-composite steel hybrid girders which are braced to develop the full flexural resistance. The provisions of Clauses 10—9 and 10—10 shall be met except as modified by this clause.

10 — 12.1.1 **Non-Composite Hybrid Girder**

A non-composite hybrid girder shall have both flanges of the same specified minimum yield stress and a web with a lower specified minimum yield stress. The compression flange area shall be equal to the tension flange area or greater by an amount not exceeding 25%.

10 — 12.1.2 **Composite Hybrid Girder**

A composite hybrid girder shall have a tension flange with a higher specified minimum yield stress than the web and a compression flange with a specified minimum yield stress greater than or equal to that of the web. In the positive moment region, the compression flange area shall be equal to the tension flange area or smaller by an amount not exceeding 25%.

In the negative moment region, the compression flange area shall be equal to the tension flange area or greater by an amount not exceeding 25%.

10 — 12.1.3 **Orthotropic Hybrid Girder**

An orthotropic hybrid girder with an orthotropic deck as the top flange shall have a web with a lower specified minimum yield stress than the bottom flange.

10 — 12.1.4 **Web Yield Stress**

The specified minimum yield stress of the web shall not be less than 35% of the specified minimum yield stress of the tension flange nor in the case of orthotropic girders less than 50% of the specified minimum yield stress of the bottom flange in regions of negative moment.

Moment Resistances

10 — 12.2

10 — 12.2.1 **Non-Composite Sections**

The factored moment resistance of non-composite non-compact sections shall be taken as: $M_r = \phi R F_{yf} S$

where for symmetrical sections,

$$R = \frac{12 + \beta(3\rho - \rho^3)}{12 + 2\beta}$$

and for unsymmetrical sections,

$$R = 1 - \frac{\beta\psi(1-\rho)^2(3 - \psi + \rho\psi)}{6 + \beta\psi(3 - \psi)}$$

where: ρ = the specified minimum yield stress of the web divided by the specified minimum yield stress of the tension flange,

β = the area of the web divided by the area of the tension flange,

ψ = the distance from the outer edge of the tension flange to the neutral axis divided by the depth of the steel section.

For sections not meeting the lateral support or lateral bracing requirements of Clause 10 — 9.3.2, the factored moment resistance shall be computed in accordance with Clause 10 — 9.3.3, except that the value of M_y shall be taken as $RF_{yf}S$.

10 — 12.2.2 **Composite Sections**

The factored moment resistance as limited by flange yielding of composite sections, shall be taken as:

$$M_r' = RM_r$$

where: $M_r = \phi F_{yf} S_b$

and S_b = section modulus of the transformed section for positive moment regions and of the effective steel section for negative regions, referred to the bottom flange.

R is computed as defined in Clause 10 — 12.2.1 for unsymmetrical sections and ψ is computed with reference to the neutral axis of the transformed section in the positive moment region and the total steel section including continuous slab reinforcement in the negative moment region.

10 — 12.2.3 **Orthotropic Girders**

The factored moment resistance of orthotropic girders as limited by bottom flange yielding shall be taken as:

$$M_r' = RM_r$$

where: $M_r = \phi F_{yf} S_b$

S_b = elastic section modulus of the section referred to the bottom flange.

R is computed as defined in Section 10 — 12.2.1 for unsymmetrical sections except that ρ, β, and ψ are referred to the bottom flange rather than the tension flange.

Shear Resistance

10 — 12.3

The shear resistance of hybrid beams and girders shall be taken as:

$$V_r = \phi A_w F_s$$

where: $$F_s = \frac{290\sqrt{F_y k_v}}{h/w} \leq 0.58 F_y$$

F_y = yield stress of the web

k_v = shear buckling coefficient as defined in Clause 10 — 9.5.

Alternatively, the shear resistance may be computed in accordance with Clause 10 — 9.5 taking into account the post-buckling strength of the web in transversely-stiffened girders.

Horizontally-Curved Girders — 10 — 13

General **10 — 13.1**

This clause pertains to bridge superstructures composed of curved-in-plan steel members, employing fabricated I-shaped sections.

The provisions of Clauses 10 — 9 and 10 — 10 shall be met, except as modified by this clause.

Design Theory **10 — 13.2**

The bending, shear, and torsional force effects on which the proportioning of the individual members is based shall be derived from an analysis of the entire superstructure in accordance with Section 3 which takes into account the complete distribution of loads to the various members. The effects of curvature and lateral bending shall be taken into account.

Flanges **10 — 13.3**

Flanges of curved girders shall be proportioned to meet the requirement:

$$\frac{M_f}{M_r} + \frac{M_{ff}}{M_{rf}} \leq 1.0$$

The stability of the compression flange shall be taken into account.

Webs **10 — 13.4**

In proportioning web plates having a ratio of d_0/R greater that 0.02, curvature effects shall be considered.

Curved Composite Girders **10 — 13.5**

In determining the number of shear connectors, the effects of curvature and torsion on the connectors shall be taken into account.

Intermediate Diaphragms **10 — 13.6**

Sufficient intermediate transverse diaphragms shall be provided between the longitudinal members to distribute torsional effects to the individual members.

Fabrication **10 — 13.7**

The flanges of horizontally-curved girders shall be flame cut to the required radius. The web may be cold formed to the required curvature. Rolled beams and welded I-sections shall not be heat curved unless approved.

Trusses — 10 — 14

General **10 — 14.1**

Unless otherwise specified in Clause 10 — 14, the requirements of all other relevant clauses of Section 10 shall be satisfied.

Main truss members shall be symmetrical about the central plane of the truss.

The alignment of members shall be in accordance with the requirements of Clause 10 — 16.1.1.

The fabricated length of truss members shall be such that the resulting camber is in accordance with the requirements of Clause 10 — 6.6.

In the design of the connections of tension and compression members, shear lag shall be taken into account, in accordance with Clause 10 — 7.1.4.

Built-Up Members **10 — 14.2**

10 — 14.2.1 **General**

If the centroidal axis of a built-up section does not coincide with the corresponding axis of the truss, the effects of eccentricity shall be considered. Preferably, the components shall be connected by solid webs. Plates provided solely for the purpose of battening shall be ignored in computing the radius of gyration of the section.

10 — 14.2.2 **Tie Plates**

10 — 14.2.2.1 The separate components of tension members composed of shapes shall be connected by tie plates. The length of end tie plates shall not be less than 1.25 times the distance between the inner lines of fasteners or welds connecting them to the flanges. Intermediate tie plates shall have a length at least 0.5 times the distance between the inner lines of fasteners or welds connecting them to the flanges. The clear distance between tie plates shall not exceed 15 times the width of the flange to which they are attached.

10 — 14.2.2.2 Tie plates shall have a minimum thickness of not less than 0.02 of the clear distance between fasteners or welds connecting them to the flanges in the case of main members and not less than 0.017 of that distance in the case of bracing members.

10 — 14.2.2.3 A diaphragm shall be provided between gusset plates engaging main members if the end tie plate is 1200 mm or more from the point of intersection of the members.

10 — 14.2.3 **Perforated Cover Plates**

10 — 14.2.3.1 The thickness of perforated cover plates shall be not less than the clear unsupported width multiplied by $\sqrt{F_y}/840$.

10 — 14.2.3.2 Perforated cover plates shall be proportioned such that the following requirements are satisfied:

a) The transverse distance from the edge of the perforation to the nearest line of connecting fasteners or welds shall not exceed 12 times the thickness of the plate.
b) The length of perforation in the direction of load shall not exceed twice the width.
c) The clear distance between perforations in the direction of load shall be not less than the clear distance between the inner lines of connecting fasteners or welds.
d) The clear distance between the end perforation and the end of the plate, or the end of half perforation, shall be not less than 1.25 times the clear distance between the inner lines of connecting fasteners or welds.
e) The periphery of the perforation at all points shall have a minimum radius of 10 mm.
f) In compression members, the ratio of the length of the perforation to the radius of gyration of the half-member at the centre of the perforation, about its own axis shall be not more than 20; nor more than 0.33 of the slenderness ratio of the member about its axis perpendicular to the plane of the plate.
g) The perforated plate shall be connected to the main components so that the solid areas of the plate, extended at 45° from the edges of the perforations, can resist the moment which would be specified for an equivalent batten plate.

10 — 14.2.4 **Battens for Compression Members**

10 — 14.2.4.1 Battened compression members may be used if the member does not carry calculated bending moments causing bending in the plane of the battens in addition to axial force.

10 — 14.2.4.2 The spacing of battens shall be such that the following requirements are satisfied:

a) If the slenderness ratio of the member about the axis perpendicular to the battens is equal to, or less than, 0.8 times the slenderness ratio of the member about the axis parallel to the battens, then the spacing of battens centre-to-centre of end fasteners or the clear distance between welds shall be such that the slenderness ratio of either main component over that distance shall not exceed 50 or 0.7 times the slenderness ratio of the member about the axis parallel to the battens.
b) If the slenderness ratio of the member about the axis perpendicular to the battens exceeds 0.8 times the slenderness ratio of the member about the axis parallel to the battens, then the spacing of battens centre-to-centre of end fasteners or the clear distance between welds shall be such that the slenderness ratio of either main component over that distance shall not exceed 40 nor 0.6 times the slenderness ratio of the member as a whole about its weaker axis.

10 — 14.2.4.3 Battens may be plates, channels, or beam sections and shall be bolted or welded to the main components so as to resist simultaneously a longitudinal shear force of:

$$V_f = \frac{.025\,C_f d}{na}\text{, and a moment}$$

$$M_f = \frac{.025\,C_f d}{2n}$$

where: d = the longitudinal distance centre-to-centre of battens
n = number of parallel planes of battens.

10 — 14.2.4.4 Battens shall have an effective length not less than the distance between the innermost connecting bolts, or welds, but in no case shall the effective length of a batten be less than twice the width of one main component in the plane of the batten. The effective length of a batten shall be taken as the longitudinal distance between end bolts or end welds, or the length of continuous welds.

10 — 14.2.4.5 Batten plates shall have a thickness of not less than 0.02 of the minimum distance between the innermost lines of connecting bolts, or welds. Batten plates with stiffened edges, or rolled shapes with flanges perpendicular to the main components, shall not be subject to this requirement.

Bracing **10 — 14.3**

10 — 14.3.1 Through truss spans, deck truss spans, and spandrel braced arch spans shall have top and bottom lateral bracing systems.

10 — 14.3.2 The lateral bracing of compression chords preferably shall be as deep as the chords and effectively connected to both flanges of the chords.

10 — 14.3.3 Through truss spans shall have portal bracing, preferably of the double plane or box type, rigidly connected to the end post and top chord flanges, and as deep as the clearance will allow. If single plane portal bracing is used it shall be located preferably in the central transverse plane of the end posts, with diaphragms between the end post webs to provide for force distribution. Portal bracing shall be proportioned to take the full reaction of the top chord lateral system and the end posts shall be proportioned to transfer this reaction to the truss bearings. If required, sway bracing shall be located at such panel points as necessary and shall be kept as shallow as practicable and of depth not less than that of the top chord.

10 — 14.3.4 Deck truss spans shall have sway bracing in the plane of the end posts. Unless an analysis performed in accordance with Section 3 indicates otherwise, sway bracing shall be provided at all intermediate panel points. This bracing shall extend the full depth of the trusses below the floor system. The end sway bracing shall be proportioned to carry the entire upper lateral forces to the supports through the end posts of the truss.

10 — 14.3.5 In the absence of a detailed analysis, in half-through or pony truss spans the vertical truss members, floor beams and connections between them shall be proportioned to resist a lateral force at ultimate limit state of not less than 8 kN/m applied at the top chord panel points of each truss.

10 — 14.3.6 The top chord shall be considered as a column having elastic lateral supports at each panel point. The critical buckling load of the column, so determined, shall be at least equal to the maximum force in any panel of the top chord resulting from loads at the ultimate limit state.

Orthotropic Decks — 10 — 15

General — 10 — 15.1

This clause pertains to the design of steel bridges which utilize a stiffened steel plate as a deck.

Design Theory — 10 — 15.2

The effects of distortion of the cross-sectional shape due to torsion shall be taken into account in analysing the girders of orthotropic box-girder bridges.

Effective Width of Deck Plate — 10 — 15.3

10 — 15.3.1 **Ribs and Beams**

The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam shall be calculated by a rational method.

10 — 15.3.2 **Girders**

The full width of deck plate may be considered effective as the top flange of the girders if the effective span of the girders is not less than:

a) 5 times the maximum distance between girder webs; and
b) 10 times the maximum distance from the edge of the deck to the nearest girder web.

The effective span shall be taken as the actual span for simple spans and the distance between points of dead load contraflexure for continuous spans. Alternatively, the effective width may be determined by appropriate analytical methods in accordance with Section 3. The effective width of the tension flange of a box girder shall be determined in accordance with Clause 10 — 11.

Moment Resistance — 10 — 15.4

10 — 15.4.1 **Local Bending In Deck Plate**

For the purposes of this clause, 'local bending' means the action of the deck plate required to carry a specified wheel load to the ribs and beams.

The local transverse factored moment resistance of the deck plate shall be taken as not greater than:

$$M_{rp} = 1.10 \phi F_y S_p$$

unless a detailed fatigue analysis or applicable fatigue tests justify a greater moment resistance, where:

S_p = section modulus of a deck plate based on the effective width of the plate.

For deck configurations in which the spacing of transverse beams is at least 3 times the spacing of longitudinal rib webs, the effects of local longitudinal and transverse bending in the deck plate need not be combined with the effects of other bending referred to in Clause 10 — 15.4.2.

10 — 15.4.2 **Longitudinal Ribs, Transverse Beams, and Longitudinal Girders**

The factored moment resistances of longitudinal ribs, transverse beams and longitudinal girders calculated in accordance with Clause 10 — 9 shall be equal to or greater than moment caused by factored loads.

For the combined bending of the longitudinal rib and longitudinal girder the factored moment resistance shall be such that:

$$\frac{M_{fr}}{M_{rr}} + \frac{M_f}{M_r} \leq 1.33$$

Slenderness Requirement of Longitudinal Ribs

10 — 15.5 The slenderness ratio of longitudinal ribs shall be adequate to ensure that overall buckling of the deck will not occur as a result of compression induced by bending of the longitudinal girders.

Deflection

10 — 15.6 The deflection of the deck plate at the serviceability limit state Type II due to live load and the dynamic load allowance shall not exceed 0.0033 of the distance between webs of ribs, and the deflection of ribs and transverse beams shall not exceed 0.002 of their span.

Diaphragms

10 — 15.7 Diaphragms or cross frames shall be provided at each support and shall be of sufficient stiffness and strength to transmit lateral forces to the bearings and to resist transverse rotation, displacement and distortion.

Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders.

Design Detail Requirements

10 — 15.8

10 — 15.8.1 Closed ribs shall be continuous through the holes cut in the transverse beams and shall be sealed against entrance of moisture by continuous welds or by seal plates.

10 — 15.8.2 Main girder transverse intermediate stiffeners shall be cut short of the deck plate a distance of 4w.

Splices and Connections

10 — 16

General

10 — 16.1

10 — 16.1.1 **Alignment of Axially-Loaded Members**

If axially-loaded members joined at their ends do not have their centroidal axes intersecting at a common point, the effect of connection eccentricity shall be included in the determination of member and connection resistance.

10 — 16.1.2 **Connection or Splice Placement**

Connections or splices shall be balanced about the axes of the connected members insofar as is practicable. Disposition of fillet welds to balance the forces transmitted by the welds about the centroidal axis of single angle, double angle, or similar types of axially-loaded members is not required. Eccentricity between the axes of such members and the gauge lines of bolted end connections also may be neglected.

Connections of axially-loaded members subject to cyclic loading shall satisfy the requirements of Section 11.

10 — 16.1.3 **Tension and Compression Member Splices and Connections**

At the ends of tension members and compression members not finished to bear, the connection shall be proportioned to resist the factored loads at the ultimate limit state. The connection also shall be proportioned to resist at least 75% of the factored resistance of the smaller of the connected members based on the behaviour of the members in the overall structure unless otherwise approved. No member nor component part except handrails or fillers shall be connected with fewer than two bolts.

At the ends of compression members which are finished to bear, the splice material and connecting bolts or welds shall be arranged to hold all parts in place and shall be proportioned for 50% of the loads at ultimate limit state.

10 — 16.1.4 **Beam and Girder Connections**

End connections for beams and girders, which are proportioned for reaction shear only, shall be detailed to reduce the flexural and restraint to the minimum amount practicable. Inelastic action in the connection at a serviceability limit state Type II load level is permitted in order to accommodate the end rotations of unrestrained simple beams.

10 — 16.1.5 **Splices and Moment Connections**

Beams and girders subject to both reaction shear and end moment due to rigid, continuous or cantilever construction, shall have connections designed at ultimate limit state for the combined effect of shear, bending and, if present, axial forces.

Bolted splices and bolted connections shall be proportioned to resist at least 75% of the factored resistance of the smaller of the connected members, based on the behaviour of the members in the overall structure, unless otherwise approved. All welded splices shall be made with complete joint penetration groove butt welds.

Where cyclic distortion of end connections occurs, the requirements of Section 11 shall be satisfied.

Bolts **10 — 16.2**

10 — 16.2.1 **Bolts In Tension**

10 — 16.2.1.1 **Tensile Resistance At Ultimate Limit State**

The factored tensile resistance, T_r, developed by the bolts in a bolted joint subject to tension, T_f, shall be taken as:

$$T_r = 0.75\phi_b n A_b F_u$$

where F_u is the specified ultimate strength of the bolt material

Bolts required to resist tension shall be proportioned to resist, independently of bolt pretension, the factored tensile force, T_f, taken as the sum of the factored external load and any additional tensile load caused by deformation of the connected parts

10 — 16.2.2 **Bolts In Shear**

10 — 16.2.2.1 **Shear Resistance At The Serviceability Limit State Type II**

Connections shall be proportioned to resist forces and moments caused by the loads at the serviceability limit state Type II. The slip resistance V_s shall be taken as

$$V_s = 0.26\,\mu\, m\, n\, A_b F_u$$

where F_u is the ultimate strength of the bolt material

Values of μ for several surfaces are given in Table 10 — 16.2.2.1

Table **Values of μ** **10 — 16.2.2.1**

Category	Steel Surface Treatment	A325 Bolts	A490 Bolts
A	Tightly-adhering clean mill scale except for quenched and tempered steels	0.59	0.51
B	Blast cleaned steels except for quenched and tempered steels	0.99	0.87
C	Grit-blasted, quenched and tempered steels	0.69	0.60
D	Hot-dip galvanized	0.31	0.27
E	Galvanized, wire brushed or blasted or zinc metallized	0.76	0.66

10 — 16.2.2.2 **Shear Resistance At Ultimate Limit States**

The shear resistance of a bolted joint subjected to shear, V_f, shall be taken as the lesser of the following:

- bearing resistance, B_r

  where $B_r = \phi_b t n e F_u \leq 3\,\phi_b t n d F_u$

  where F_u is the ultimate strength of the plate.
- shear resistance, V_r

  where $V_r = 0.60\,\phi_b\, n m A_b F_u$ where ϕ_b is equal to 0.67 and F_u is the ultimate strength of the bolt material

If the bolt threads are intercepted by any shear plane, the factored shear resistance of the joint shall be taken as 70% of V_r. In a tension splice in which the bolt line is longer than 1.3 m, the resistance shall be taken as 80% of the calculated resistance.

10 — 16.2.3 **Bolts in Shear and Tension**

10 — 16.2.3.1 **Resistance At Serviceability Limit States Type II**

A connection subjected to a load which causes shear, V, and a tension, T, on the bolts, shall satisfy the following relationship:

$$\frac{V}{V_s} + 1.9\frac{T}{nA_bF_u} \leq 1.0$$

where V_s = shear resistance as defined in Clause 10 — 16.2.2.1, except that the tension T_f at the ultimate limit state shall not exceed the value of T_r as given in Clause 10 — 16.2.1.1.

10 — 16.2.3.2 **Resistance At Ultimate Limit States**

A bolt which is required to develop resistance to a tensile force and a shear force which results from loads at the ultimate limit state shall satisfy the following relationship:

$$V_f^2 + \beta T_f^2 \le 0.56\phi_b^2\,\beta(A_bF_u)^2$$

where: β = 0.69 for A325 bolts, shear plane through shank
= 0.41 for A325 bolts, shear plane through threads
= 0.56 for A490 bolts, shear plane through shank
= 0.30 for A490 bolts, shear plane through threads

and the factored loads V_f and T_f are calculated on a per bolt basis, except that V_f shall not exceed V_r or B_r associated with the bolt as given in Clause 10 — 16.2.2.2.

Welds **10 — 16.3**

10 — 16.3.1 Welding design and practice shall conform to CSA Standard W59, *Welded Steel Construction (Metal Arc-Welding)*, except as modified by this clause. The factored resistance of weldments shall be calculated in accordance with the provisions set forth in Table 10 — 16.3.1 in which V_r shall be taken as the smaller of:

a) $V_r = 0.66\,\phi A_m F_y$ for base metal, and

b) $V_r = 0.50\,\phi A_w X_u$ for weld metal.

The vector sum of loads at the ultimate limit state shall not exceed the factored resistances calculated in accordance with Table 10 — 16.3.1.

Table **Factored Resistance of Welds** **10 — 16.3.1**

Type of Weld	Type of Load and Orientation	Factored Resistance
Complete Penetration Groove Welds	Tension or compression parallel to weld axis (no load transfer).	Same as for base metal
	Tension or compression normal to effective throat.	
	Shear on effective throat.	V_r
Partial Penetration Groove Welds	Tension or compression parallel to weld axis (no load transfer).	Same as for base metal.
	Compression normal to effective throat.	
	Tension inclined to weld axis.	V_r
	Shear on effective throat.	
Fillet Welds	Tension or compression parallel to weld axis (no load transfer).	Same as for base metal
	Tension or compression other than parallel to weld axis.	V_r
	Shear on effective throat.	

10 — 16.3.2 Except as otherwise permitted in this section, the eccentricity of the vector sum of loads shall be accounted for in the calculation of weld resistance.

Connection and Splice Details **10 — 16.4**

10 — 16.4.1 Bolted parts shall fit together solidly when assembled and shall not be separated by gaskets or any other interposed compressible material.

10 — 16.4.2 Bolt holes may be punched, sub-punched or sub-drilled and reamed, or drilled. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size except that, where shown in the plans and at other approved locations, enlarged or slotted holes may be used with high-strength ASTM A325 or A490 bolts 5/8 inches in diameter and larger, or with ASTM A325M or A490M bolts 16 mm in diameter and larger.

Hardened washers shall be used under the head and the nut of bolts in such oversize holes.

The slots may be used without regard to direction of loading.

10 — 16.4.3 Faying surfaces of bolted joints in categories A, B or C shall be free of oil, paint, lacquer, or other coatings.

Faying surfaces in category E may be coated by:

a) hot-dip galvanizing, providing that the faying surfaces are wire-brushed or 'brush-off' blast cleaned after galvanizing and prior to assembly; or

b) zinc coatings applied in accordance with CSA Standard G189, *Sprayed Metal Coatings for Atmospheric Corrosion Protection*; or

c) other materials and methods provided that these have been sufficiently tested to establish the slip performance of full-size similarly coated or treated joints, and have been approved.

10 — 16.4.4 The minimum distance between centres of bolt holes preferably shall be not less than 3 bolt diameters and in no case less than 2.67 diameters. The minimum distance from the centre of a bolt hole to any edge shall be that given in Table 10 — 16.4.4.

The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part, with a maximum of 150 mm.

In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance measured from the centre of the end fastener to the nearest end of the connected part, shall be not less than the edge distance values given above. In members having either one or two bolts in the line of load, the end distance shall be not less than 1.5 bolt diameters.

Maximum and minimum edge distance requirements for bolts in slotted or oversize holes shall apply by assuming that the fastener can be placed at any extremity of the slot or hole.

Table **Minimum Edge Distance for Bolt Holes** **10 — 16.4.4**

Bolt Designation and Diameter (inches)	At Sheared Edge	At Rolled or Gas-Cut Edge**
M16 ⅝	28 mm	22 mm
M20 ¾	32 mm	25 mm
M22 ⅞	38 mm*	28 mm
M24 1	44 mm*	32 mm
M27 1⅛	51 mm,	38 mm
M30 1¼	57 mm	41 mm
M36 (over 1¼)	1.75 × diameter	1.25 × diameter

* At ends of beam framing angles this distance may be 32 mm.

**Gas-cut edges shall be smooth and free from notches. Edge distance in this column may be decreased 3 mm when the hole is at a point where computed stress under factored loads is not greater than 0.3 times the yield stress.

Connection Reinforcement and Stiffening — 10 — 16.5

10 — 16.5.1 **Gusset Plates**

Gusset or connection plates with re-entrant cuts other than smooth curves for appearance shall not be used.

10 — 16.5.1.1 **Edge Stiffeners**

The unsupported edge of a gusset plate shall be stiffened if the length exceeds $930/\sqrt{F_y}$ times the thickness.

10 — 16.5.1.2 **Shear Resistance**

The factored shear resistance of the gross area of gusset plates shall be taken as:

$$V_r = 0.5\,\phi A_g F_y$$

10 — 16.5.2 **Moment Connections**

For beams rigidly framed to the flange of an H-type member, stiffeners shall be provided on the web of the H-type member as follows:

opposite the compression flange of beam when

$$B_r = \phi w_c (t_b + 5k)/F_{yc} < \frac{M_f}{d_b}$$

except that for H-members with non-compact webs,

$$B_r = \phi \frac{640\,000}{(h_c/w_c)^2} w_c (t_b + 5k)$$

opposite the tension flange of beam when

$$T_r = 7\phi F_{yc} t^2 < \frac{M_f}{d_b}$$

The stiffener or pair of stiffeners opposite either beam flange shall develop a force equal to:

$$F_{st} = \frac{M_f}{d_b} - B_r$$

at the ultimate limit state.

In all cases the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection.

When beams frame to one side of a column only, the stiffeners need not be longer than half the depth of the column.

Pins, Rollers, Rockers, and Bearing Plates — 10—17

Bearing Resistance — 10—17.1

The factored bearing resistance, B_r, developed by a member or portion of a member, subjected to bearing shall be as follows.

a) on the contact area of machined, accurately sawn or fitted parts and on the bearing area of pins not subject to rotation,

$$B_r = 1.50\,\phi F_y A$$

where the bearing area of pins is taken as the pin diameter multiplied by the thickness of the connected parts.

b) on the bearing area of pins subject to rotation or of pins likely to be removed and replaced,

$$B_r = 0.75\,\phi F_y A;$$

c) on expansion rollers or rockers,

$$B_r = 0.00013\,\phi D L F_y^2$$

where F_y is the specified minimum yield point of the weaker part in contact.

Pins — 10—17.2

10—17.2.1 The diameter of the pins in a connection containing eyebars shall be not less than 0.8 of the width of the widest bar.

10—17.2.2 Pins shall be of sufficient length to ensure full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers. If the pins are bored, through rods with recessed cap washers may be used. Pin nuts shall be malleable steel castings; they shall be secured either by cotter pins in the screw ends or by providing for screw ends which are long enough to permit burring the threads.

10—17.2.3 Members shall be held against lateral movement on the pins.

10—17.2.4 Pins shall be so located with respect to the centroidal axes of the members as to reduce stresses due to bending to a minimum.

10—17.2.5 Pin plates shall be of sufficient thickness to provide the required bearing area upon the pin; they shall be as wide as the dimension of the member will allow; and their length, measured from pin centre to end, shall be at least equal to their width. Pin plates shall contain sufficient fasteners to distribute their due portion of the pin load to the full cross-section of the member, and only fasteners located in front of two lines drawn from the centre of the pin towards the body of the member and inclined at 45° to the axis of the member shall be considered effective for this purpose. For members composed of web plates and flange angles, with or without cover plates, there shall be at least one outside pin plate covering the vertical legs of the flange angles.

Bearing Plates — 10—17.3

10—17.3.1 **Finishing**

Bearing plates shall be finished in accordance with the following requirements.

a) Steel-to-steel contact surfaces of rolled steel bearing plates shall be finished in such a manner that the requirements of Clause 10—21 are met. Rolled steel bearing plates 50 mm or less in thickness may be used without planing provided that a satisfactory contact bearing is obtained; rolled steel bearing plates over 50 mm but not over 100 mm in thickness may be straightened by pressing or by planing on all bearing surfaces to obtain a satisfactory contact bearing; rolled steel bearing plates over 100 mm in thickness shall be planed on all bearing surfaces except as noted in (c) below.

b) Column bases other than rolled steel bearing plates shall be planed on all bearing surfaces except as noted in (c) below.

c) The bottom surfaces of bearing plates and column bases which rest on concrete foundations and are grouted to ensure full bearing need not be planed.

Torsion Members 10—18

General 10—18.1

The factored torsional resistance, Q_r, developed by a member subjected to torsional moments shall be determined in accordance with this clause.

Members of Closed Cross-Section 10—18.2

10—18.2.1 The factored torsional resistance, Q_r, shall be computed from:

$$Q_r = 1.15\,\phi F_y A' t$$

where:
A' = area enclosed by the median line of the wall
t = minimum thickness of material

10—18.2.2 The width-to-thickness ratios of elements shall be such as to permit the attainment of yield under the action of factored loads and also such that

a) for flat plate elements $b/t \leq 1100/\sqrt{F_y}$
b) for circular hollow sections $D/t \leq 18\,000/F_y$

10—18.2.3 **Combined Bending and Torsion**

Members of closed cross-section subject to both bending and torsional moments shall be proportioned so that the maximum principal stress does not exceed ϕF_y.

10—18.2.4 **Reinforcement of Cut-Outs**

Members of closed cross-section with cut-outs whose torsional design is based on the cross-sectional properties of the closed cross-section shall be designed as follows:

a) Cut-outs shall have semi-circular ends.
b) The width of a cut-out shall not exceed 0.17 of the circumference of the member.
c) A stiffener shall be provided around the perimeter of the cut-out and welded to develop the full cross-section of the wall; such stiffener shall have a cross-sectional area, A, given by $A = Lt/\sqrt{3}$ where L is the length of the cut-out measured parallel to the longitudinal axis of the member, and t is the thickness of the wall of the member.
d) The b/t ratio of the outstanding portions of the stiffener shall not exceed $170/\sqrt{F_y}$.

Members of Open Cross-Section 10—18.3

10—18.3.1 The factored torsional resistance, Q_r, shall be computed from:

a) when $b/t \geq 7$; $Q_r = \Sigma 0.29\phi F_y b_i t_i^2$

b) when $b/t < 7$; $Q_r = \Sigma 0.19\phi F_y b_i t_i^2$

where Σ = sum for all elements,
t_i = thickness of the ith element, and
b_i = length of the ith element.

10—18.3.2 The width-to-thickness ratios of elements shall be in accordance with Clause 10—18.2.2.

Members of Doubly-Symmetric I-Shaped Section 10—18.4

10—18.4.1 **Warping Restraint Present**

The factored torsional resistance, Q_r, shall be computed from:

$$Q_r = Q_{r1} + Q_{r2}$$

where: $Q_{r1} = Q_r$ as given in Clause 10—18.3, and

$$Q_{r2} = \frac{\phi F_y t b^2 d'}{4\ell}$$

where: $d' = d - t$

ℓ = length in which warping restraint is developed.

10—18.4.2 **Combined Bending and Torsion**

Members subject to both bending and torsional moments shall be proportioned so that:

$$\left[\frac{M_f}{M_r}\right]^2 + \left[\frac{Q_f}{Q_r}\right]^2 \leq 1.0 \text{ and}$$

the stability of flanges under flexural forces and lateral bending due to torsion shall be ensured.

Standardized Truss Systems

10—19

General

10—19.1

10—19.1.1 This clause pertains to all standardized truss systems composed of prefabricated steel panels and connecting members and assembled at the construction site.

10—19.1.2 The load-carrying capacity and fatigue strength shall be defined for each member individually of the standardized truss system for the various specified supporting and bracing conditions. Such information for each standardized product offered by the manufacturer shall be determined by analysis or load testing and shall be subject to approval.

For the assembled truss system, the load-carrying capacity shall be based on the strengths and properties of the individual components. Special consideration shall be given to problems of buckling and overall stability of the system, taking into account deviations from the specified supporting and bracing conditions of individual panels and members due to tolerances of assembly and their cumulative effect on deflections.

10—19.1.3 Unless otherwise specified in Clause 10—19, all of the relevant requirements of Section 10 shall be satisfied.

Analysis

10—19.2 Analysis of standardized truss systems and their individual components shall be carried out in accordance with the provisions of Section 3.

Materials

10—19.3 Materials of the various components of the standardized truss systems shall meet the requirements of Clause 10—4.

Fatigue

10—19.4 The fatigue requirements of Section 11 shall be met.

Design Detail Requirements

10—19.5 The requirements of Clause 10—6 shall be met. Flame-cut holes or slots in tension members shall have a minimum distance of 10 mm from the edge of any weld.

Load Tests

10—19.6 Load Tests on components and assemblies of standardized truss systems if undertaken in accordance with Clause 10—19.1.2 shall be carried out in accordance with the requirements of Section 14.

Piles

10—20

Steel Piles

10—20.1 The design of steel piles shall be in accordance with the requirements of Section 6 and Section 10.

Effective Length

10—20.2 The effective length, KL, of a pile shall be determined on the basis of the resistance provided by the soil, the pile cap, and the superstructure.

Splices

10—20.3 Splices shall develop the axial compressive strength, C_y, of the pile at yield stress.

Composite Tube Piles

10—20.4

10—20.4.1 The factored structural resistance of a pile consisting of a steel tube filled with concrete shall be taken as:

$$C_{rc} = C_r + C'_r$$

where: $C'_r = F_c A_c$, and

F_c shall be computed as follows:

for $\frac{KL}{r} \leq \sqrt{\frac{\pi^2 E}{F_y}}$

$$F_c = 0.85\,\phi_c f'_c \left[\frac{\sqrt{\frac{\pi^2 E}{F_y}} - \frac{KL}{r}}{\sqrt{\frac{\pi^2 E}{F_y}}} \right]$$

for $\frac{KL}{r} > \sqrt{\frac{\pi^2 E}{F_y}}$; $F_c = 0$

where $\phi_c = 0.67$ and the slenderness ratio shall be taken as that of the steel section alone.

10—20.4.2 Where bending as well as axial compression is to be resisted, the bending shall be assumed to be resisted by the steel section alone. The steel section shall be proportioned as a beam column to carry the total bending, plus axial compression, C_f.

---

abrication, ransportation, rection and Inspection 10—21

General **10—21.1** Unless otherwise specified, the provisions of Clause 10—21 shall apply and fabrication and erection shall also conform to the requirements of CSA Standard CAN3-S16.1-M78, *Steel Structures for Buildings—Limit States Design*.

The designer shall indicate on the contract documents the provisions with respect to fabrication, transportation, erection and inspection that the contractor is required to follow.

Welded Construction **10—21.2**

10—21.2.1 Unless otherwise specified, welding design and practice shall conform to CSA Standard W59, *Welded Steel Construction (Metal Arc-Welding)* and the requirements of Section 11.

10—21.2.2 **Fabricator and Erector Qualification**

In order to assume full responsibility for welded construction, fabricators and erectors shall be certified by the Canadian Welding Bureau in accordance with the requirements of CSA Standard W47-1, *Certification of Companies for Fusion Welding of Steel Structures*, for Division 1 or for Division 2. They shall have the organization, personnel, welding procedures and equipment required to produce satisfactory welds and weldments consistent with good bridge practice.

Transportation **10—21.3** Structural components shall be transported in such a manner as to avoid the development of fatigue cracks.

Erection **10—21.4** The erector shall ensure during erection that an adequate resistance in the uncompleted structure is provided using loads in accordance with Section 2.

Bolted Construction **10—21.5** Bolted construction shall conform to the requirements of CSA Standard CAN3-S16.1-M78, *Steel Structures for Buildings—Limit States Design*.

Unless approved otherwise, only the turn-of-nut tightening method shall be used for installation of high-strength bolts.

---

gn, uminaire and raffic Signal upports 10—22

General **10—22.1** The hollow steel sections for sign, luminaire and signal supports shall be proportioned using appropriate clauses of this section, unless otherwise specified in Clauses 10—22.2 to 10—22.4, inclusive.

Width-to-Thickness and Diameter-to-Thickness Requirements **10—22.2** The width-to-thickness and diameter-to-thickness requirements of the compression elements of hollow square, hexagonal, octagonal and dodecagonal sections shall conform to the requirements of Clause 10—8.2 and Table 10—8.2.1. For circular and multi-sided hollow sections, D is defined as the diameter of the circle which inscribes the mid-thickness of the wall of the section.

Combined Axial Compression, Flexure and Torsion **10—22.3** Sections subject to combined axial compression, flexure and torsion shall be proportioned to meet the following relationship:

$$\frac{C_f}{C_r} + \frac{M_f}{M_r} + \left[\frac{Q_f}{Q_r}\right]^2 \leq 1$$

where: C_r is as given in Clause 10—8.3,

M_r is as given in Clause 10—9, and

Q is as given in Clause 10—18.

Design Detail Requirements **10—22.4** Openings for drainage shall be provided.

11 Structural Fatigue and Fracture Control

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Scope

11 — 1

This section provides requirements for the design of steel and aluminum components and steel reinforcement subjected to repeated applications of load. The section requires that members subject to cyclic stress during passage of the highway live load or as a result of wind effects (vortex shedding) be proportioned on the basis of an allowable stress range, F_{sr}, corresponding to a specified number of stress cycles, N_D, where for highway live loads N_D is a function of the highway classification given in Section 2, Table 2 — 4.3.1.1.

Aerodynamic response of components which may result in cyclic fatigue, shall be considered in accordance with the provisions of Section 2.

Only the computed stress range, as defined herein, need be less than the allowable stress range specified for the appropriate type and location of design detail. Fracture control and fracture toughness requirements for fracture critical members and primary tension members are also provided.

Definitions

11 — 2

Fatigue — initiation of microscopic cracks and/or propagation of such cracks into macroscopic cracks caused by the repeated application of load.

Fatigue Limit — the level of stress range below which no fatigue crack growth is assumed to occur.

Fracture Critical Members — members, including attachments, in a single load path structure, which are subject to tensile stress and whose failure could lead to collapse of the structure.

Prying Action — an additional force introduced into fasteners as a result of the distortion of connected parts.

Primary Tension Members — members, including attachments, which are subject to tension stress.

Single Load Path Structure — a structure in which failure of a single component could lead to a total collapse.

Stress Range Category — a category which establishes the level of stress range permitted according to the classification of the detail and the number of design stress cycles.

Stress Range — algebraic difference between the maximum and the minimum stresses due to serviceability limit state Type I load. Tensile stress shall have the opposite sign to that of compressive stress.

Notation

11 — 3

A_{sr}	area of longitudinal slab reinforcement per beam over an interior support, mm².
d	diameter of stud shear connector, mm.
F_{sr}	allowable stress range, MPa, defined in Tables 11 — 4.1 and 11 — 9.1.
g	distance from heel of connection angle to first gauge line of bolts in outstanding legs, mm.
H	height of stud shear connector, mm.
I	moment of inertia of the transformed composite steel girder in positive moment regions and in negative moment regions which are prestressed in accordance with the provisions of Section 9, or moment of inertia provided by the steel beam including or excluding the area of reinforcement embedded in the concrete in negative moment regions, mm⁴.
L	span length between simple connections at girder ends, mm.
N_c	number of additional shear connectors per beam at point of contraflexure.
N_D	specified number of design stress cycles; i.e., the number of occurrences of design stress range which causes fatigue damage equivalent to that caused by the actual variable stress spectrum.
Q	the moment of area about the neutral axis of the composite section of the transformed compressive concrete area in positive moment regions and in negative moment regions which are prestressed in accordance with the provisions of Section 9 or, in the case of non-prestressed sections, of the transformed area of reinforcement embedded in the concrete for negative moment regions, mm³.
S_r	the range of interface shear per unit length at the interface of the slab and girder at a given point, N/mm.
t	thickness of end connection angles, mm.
V_r	the shear range, N.
w	the length of a channel shear connector on the flange of girder measured in a transverse direction, mm.
Z_r	the allowable range of interface shear on an individual shear connector, N.

Structural Steel Components, Connections and Fasteners 11—4

Allowable Stress Range and Design Stress Cycles **11—4.1**

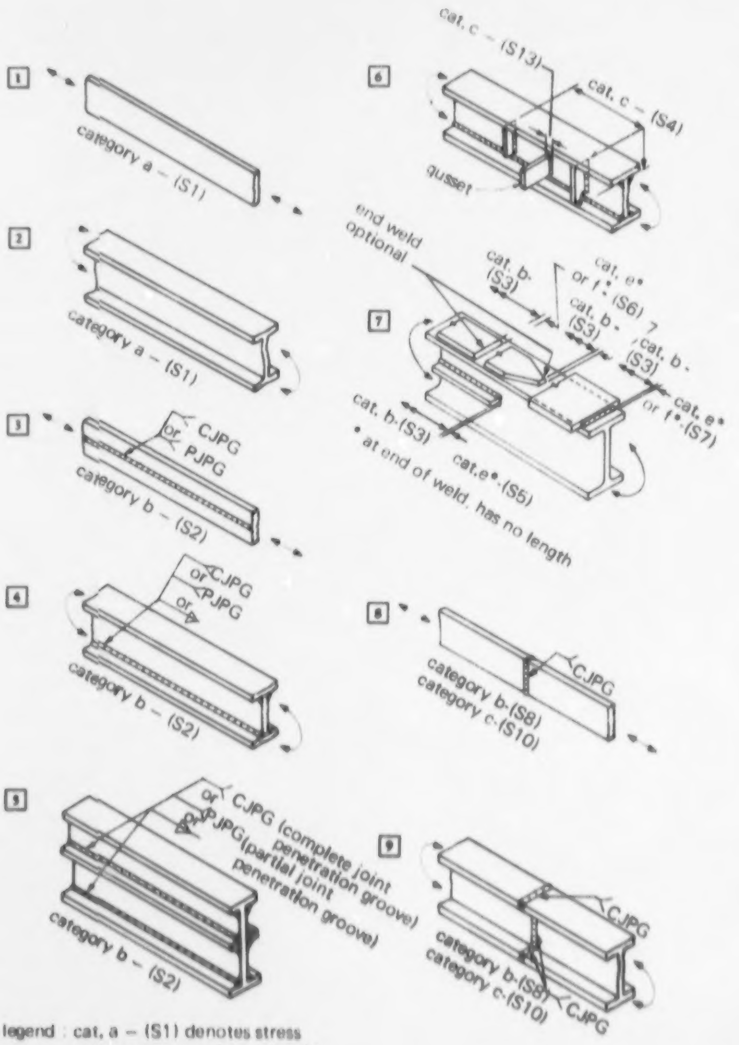
Components, connections, mechanical fasteners and welds subjected to repeated variations of tensile stress or to reversals of stress shall be proportioned so that the computed stress range does not exceed the allowable stress range, F_{sr}, specified in Table 11—4.1 for the stress category specified in Clause 11—4.2 and the highway class as given in Table 2—4.3.1.1.

When the span length exceeds 20 m, the stress range as computed for a serviceability limit state Type I loading shall be reduced by 5% prior to comparison with the allowable stress range specified in Table 11—4.1.

Stress Categories **11—4.2**

In the absence of more specific approved data, stress categories shall be as given in Table 11—4.2.

Figure **Illustrative Examples of Various Fatigue Stress Categories** **11—4.1(a)**


1
category a – (S1)
2
category a – (S1)
3
CJPG
or
PJPG
category b – (S2)
4
CJPG
or
PJPG
or
category b – (S2)
5
CJPG (complete joint penetration groove)
or
PJPG (partial joint penetration groove)
or
category b – (S2)
6
cat. c – (S13)
cat. c – (S4)
gusset
7
end weld optional
cat. b-(S3)
cat. e* or f*-(S6)
cat. b-(S3)
cat. b-(S3)
cat. e* or f*-(S7)
cat. b-(S3)
cat. e*-(S5)
* at end of weld, has no length
8
CJPG
category b-(S8)
category c-(S10)
9
CJPG
category b-(S8)
category c-(S10)
CJPG

legend : cat. a – (S1) denotes stress
range category - (situation 1)

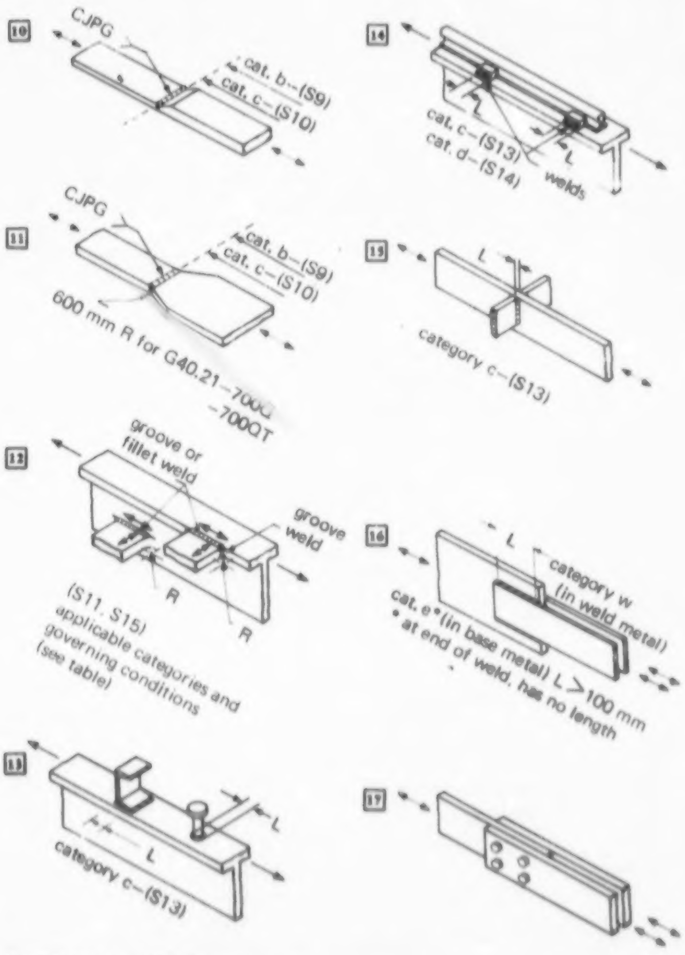
Table **Allowable Stress Range in Fatigue of Steel*** **11 – 4.1**

Stress Category in Accordance with Table 11 – 4.2	Allowable Stress Range F_{sr}, MPa			
	Class C-2 for 100 000 Cycles	Class C-1 for 500 000 Cycles	Class B for 2 000 000 Cycles	Class A over 2 000 000 Cycles
a	415	250	165	165
b	310	190	125	110
c	220	130	90	70**
d	185	110	70	48
e	145	85	55	32
f	110	65	40	18
w	115	85	65	48

* For structures with span length greater than 20 m, see Clause 11 – 4.1. For fracture critical members, see Clause 11 – 4.4.

** For welds connecting transverse stiffeners to girder webs or flanges, use 83 MPa.

Figure **11 – 4.1(b)**


10
CJPG
cat. b–(S9)
cat. c–(S10)
14
cat. c–(S13)
cat. d–(S14)
L
L
welds
11
CJPG
cat. b–(S9)
cat. c–(S10)
600 mm R for G40.21–700Q
–700QT
15
L
category c–(S13)
12
groove or
fillet weld
groove
weld
R
R
(S11, S15)
applicable categories and
governing conditions
(see table)
16
L
category w
(in weld metal)
cat. e*(in base metal) L > 100 mm
* at end of weld, has no length
13
L
L
category c–(S13)
17

legend: cat. b – (S9) denotes stress
range category – (situation 9)

Figure Notes

Applicable Categories and Governing Conditions to Example 12 **11 — 4.1(b)**

Transition Radius "R"	Longitudinal Loading Transverse Loading	Longitudinal Loading	Transverse Loading Flange			
	Fillet Welded to Web	Groove Welded				
		to Web, Flange	to Web[3]	to Flange[3]		
	Stress Range Category			Stress Range Category Based on Condition of Joint[2]		
				1	2	3,4
50 mm R ≥ 0 mm	e	e[1]	e	e	e	e
150 mm R ≥ 50 mm[4]	d	d	d	d	d	d
600 mm R ≥ 150 mm[4]	d	c	c	c	c	c
R ≥ 600 mm[4]	d	b	c	b	c	e

1) For longitudinal loading only, use Category d if detail length is between 50 mm and 12 times the plate thickness, but less than 100 mm.
2) Condition of Joint
   1 — Equal thickness of parts joined, reinforcement removed.
   2 — Equal thickness of parts joined, reinforcement not removed.
   3 — Unequal thickness of parts joined, reinforcement removed.
   4 — Unequal thickness of parts joined, reinforcement not removed.
3) Weld soundness to be established by non-destructive examination.
4) Terminal ends of welded joint to be ground smooth.

High-Strength Bolts Subjected to Tensile Cycle Loads **11 — 4.3**

High-strength bolts subjected to tensile cyclic load shall be designed for the combined external and prying forces resulting from the serviceability limit state Type I load. The following requirements apply.

a) **For Bridges on Class A and Class B Highways:**
   i) If the prying load on the bolts does not exceed 5% of the externally applied load, the applied tensile stress shall not exceed 300 MPa for A325 bolts and 375 MPa for A490 bolts.
   ii) If the prying load is equal to or exceeds 5% but is less than 15% of the externally applied load, the applied tensile stress shall not exceed 150 MPa for A325 bolts and 185 MPa for A490 bolts.

b) **For Bridges on Class C Highways:**
   i) If the prying load does not exceed 10% of the externally applied load, the applied tensile stress shall not exceed 300 MPa for A325 bolts and 375 MPa for A490 bolts.
   ii) If the prying load is equal to or exceeds 10% but is less than 20% of the externally applied load, the applied tensile stress shall not exceed 180 MPa for A325 bolts and 225 MPa for A490 bolts.

Fracture Critical Members **11 — 4.4**

For fracture critical members, the values of the allowable stress ranges shall be 75% of those specified in Table 11 — 4.1.

Table **Stress Categories** **11 — 4.2**

General Condition	Situation	Stress Category	Illustrative Example in Figure 11 — 4.1
Plain Material	(S1) Base metal with rolled or cleaned surfaces. Flame cut edges with a surface roughness not exceeding 25 μm as defined by CSA Standard B95.	a	1 2
Built-up Members	(S2) Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous complete or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	b	3 4 5
	(S3) Base metal and weld metal along the length of horizontal stiffeners and cover plates connected by continuous complete or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	b	7
	(S4) Base metal at toe of transverse stiffener welds on girder webs or flanges subjected to calculated flexural stress.	c	6
	(S5) Base metal at end of longitudinal stiffeners.	e	7
	(S6) Base metal at end of partial length welded cover plates narrower than the flange, and having square or tapered ends, with or without welds across the ends.		7
	Flange thickness ≤ 20 mm	e	
	Flange thickness > 20 mm	f	
	(S7) Base metal at end of partial length cover plates wider than the flange and having square ends with welds across the ends.		7
	Flange thickness ≤ 20 mm	e	
	Flange thickness > 20 mm	f	
Complete Joint Penetration Groove Welds	(S8) Base metal and weld metal at complete penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness is established by non-destructive examination.	b	8,9

Table **Stress Categories (Cont'd)** **11 — 4.2**

General Condition	Situation	Stress Category	Illustrative Example in Figure 11 — 4.1
	(S9) Base metal and weld metal in or adjacent to complete penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 4 in 10, with grinding in the direction of applied stress, and weld soundness established by non-destructive examination. A 600 mm curved radius transition shall be used for G40.21M-700Q and 700QT steels	b	10,11
	(S10) Base metal and weld metal in or adjacent to complete joint penetration groove welded splices, with or without transitions, involving equal widths and/or thicknesses of material or involving transitions having slopes no greater than 4 in 10 when in either case reinforcement is not removed and when weld soundness is established by non-destructive examination	c	8,9, 10,11
Groove Welded Attachments	(S11) For base metal at details attached to flanges or webs by groove welds subjected to transverse or longitudinal loading (regardless of detail length except for conditions as covered by Note (1) in the table notes for Example 12) the stress range categories shall be as shown in Figure 11 — 4.1 in the table notes for Example 12. Besides being dependent on transition radius the stress range categories in the case of flange connections subject to transverse loading are also a function of relative thickness of material and whether or not groove weld reinforcement is removed.	See Tabulation in Ex. 12 Figure 11 — 4.1(b)	12
Fillet Welded Connections	(S12) Base metal at intermittent fillet welds.	e	
	(S13) Base metal adjacent to fillet welded attachments with length, L, in direction of stress less than 50 mm and stud-type shear connectors.	c	6,14 15,13

Table **Stress Categories (Cont'd)** **11—4.2**

General Condition	Situation	Stress Category	Illustrative Example in Figure 11—4.1
	(S14) Base metal at detail attached by fillet welds subjected to longitudinal loading only when the detail length, L, in direction of stress is between 50 mm and 12 times the plate thickness, but less than 100 mm and the transition radius R is less than 50 mm.	d	14
	(S15) For base metals at details attached to webs by fillet welds subjected to transverse and/or longitudinal loading, regardless of detail length, the stress range categories shall be as shown in Figure 11—4.1 in the tabulation for the same Example –Shear stress on the throat of fillet welds shall be governed by stress range category "w"	See Tabulation in Ex. 12 Figure 11—4.1(b)	12
	(S16) Except for cover plates and details attached to webs base metal at end of detail 100 mm or longer attached by fillet welds where the length of weld is in the direction of stress.	e	16
Fillet Welds	(S17) Shear stress on throat of fillet welds.	w	16
Stud-Type Shear Connectors	(S18) Shear stress on the nominal area of stud shear connectors.	w	
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connected material.	b	17
	Base metal at net section of high-strength bolted bearing type connections.	b	17
	Base metal at net section of bolted connections other than high-strength bearing type.	d	17
Anchor Bolts and Threaded Parts	Tensile stress range on the stress area of the threaded part based on its major diameter, including effects of bending.	e	

Table **Stress Categories (Cont'd)** **11 — 4.2**

General Condition	Situation	Stress Category	Illustrative Example in Figure 11 — 4.1
High-Strength Bolts in Tension	Preloaded high-strength bolts installed in accordance with the provision of Clause 10 — 21.5 and subjected to cyclic tensile stress	See Clause 11 — 4.3	

Fracture Control 11 — 5

Identification **11 — 5.1**

The components identified as fracture critical members and primary tension members shall be clearly identified on the contract plans.

Fracture Toughness Requirements **11 — 5.2**

11 — 5.2.1

For fracture critical members, Charpy V-notch tests shall be specified on a per plate frequency, and the steel shall meet the impact energy requirements given in Table 11 — 5.2.1. For such members, the fracture control plan given in Appendix A11 — 1 shall be implemented.

Table **Impact Test Temperature* and Charpy Impact Energy Requirements for Fracture Critical Members** **11 — 5.2.1**

Grade G40.21	Minimum Average Absorbed Energy, J	Test Temperatures for Minimum Service Temperature Warmer than –40°C	Test Temperature for Minimum Service Temperature Colder than –40°C
(a) Commonly Used Steels			
260 WT	20	0	–30°C
300 WT	20	0	–30°C
350 WT and AT	27	0	–30°C
400 WT and AT	27	0	–30°C
(b) These steels may be used if approved			
480 WT and AT	27	–30°C	–45°C
700 QT	34	–30°C	–45°C

11 — 5.2.2

For primary tension members, Charpy V-notch tests shall be specified on a per heat frequency, and the steel shall meet the impact energy requirements given in Table 11 — 5.2.2.

Service Temperature **11 — 5.3**

The applicable minimum service temperature shall be the minimum daily mean temperature taken from the Appendix to Section 2, Figure A2 — 1.2.

Table **Impact Test Temperature* and Charpy Impact Energy Requirements for Primary Tension Members** **11 — 5.2.2**

Grade G40.21	Minimum Average Absorbed Energy, J	Test Temperatures for Minimum Service Temp. Warmer than –40°C	T[illegible] Temperatures for Minimum Service Temp. between – 40° and –60°C	Test Temperatures for Minimum Service Temp. Colder than –60°C
(a) Commonly Used Steels				
260 WT	20	0	–20°C	–30°C
300 WT	20	0	–20°C	–30°C
350 WT and AT	27	0	–20°C	–30°C
400 WT and AT	27	0	–20°C	–30°C
(b) These steels may be used if approved				
480 WT and AT	27	–30°C	–45°C	–45°C
700 QT	34	–30°C	–45°C	–45°C

*Indicates test temperature to be used when carrying out Charpy V-notch tests

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Composite Steel Beams, Girders and Box Girders 11 — 6

Shear Connector Requirements 11 — 6.1

Steel components of composite beams, girders and box girders shall be proportioned to satisfy the requirements of Clause 11 — 4. The stress range shall be computed by using the composite section moment of inertia consistent with the predetermined properties of the materials used.

Range of Interface Shear 11 — 6.2

The shear connectors shall be designed for fatigue in accordance with the following provisions.

The range of interface shear S_r per unit length shall be:

$$S_r = \frac{V_r Q}{I}$$

The allowable range of interface shear per connector Z_r shall be computed as follows:

- For welded studs having ratios of $H/d \geq 4$ and satisfying the requirements of Section 10:

$$Z_r = F_{sr}\frac{\pi d^2}{4}$$

- For channels with 5 mm minimum fillet welds along heel and toe:

$$Z_r = F_{sr}(6.5\,w)$$

The value of F_{sr} shall be as given in Table 11 — 4.1 for Category w.

The required pitch of shear connectors at any cross-section of a girder shall be determined by dividing the allowable range of interface shear of all connectors at that cross-section, ΣZ_r, by the interface shear, S_r, per unit length. Over interior supports of continuous beams, the pitch may be modified to avoid placing connectors at locations of high stress in the tension flange, provided that the total number of connectors remains unchanged.

Additional Connectors 11 — 6.3

When shear connectors are not provided in the negative moment region, as specified in Clause 10 — 10, the number of additional connectors required at each point of contraflexure shall be:

$$N_c = 70\,A_{sr}/Z_r$$

The additional connectors shall be centered about the point of dead load contraflexure within a length equal to one-third of the effective slab width. Field splices shall preferably be located so that they clear the connectors.

Longitudinal Reinforcement 11 — 6.4

When shear connectors are provided in the negative moment region, and longitudinal reinforcement is considered a part of the composite section as specified in Clause 10 — 10, the allowable stress range in the slab reinforcement shall be as specified in Clause 11 — 10. When the slab is prestressed, and the stress in the longitudinal reinforcement is compressive, the provisions of this Clause 11 — 6.4 need not be applied.

Secondary Stresses 11 — 7

Restraint at End Connections 11 — 7.1

Simple connections designed to permit end rotation shall be detailed to minimize restraint. The stress range developed as a consequence of such a restraint shall be considered in the design of the connections and members affected.

The end connection angles of stringers and beams shall be as thin as practicable, consistent with the strength requirements. The distance g from the heel of the connection angle to the first gauge line of bolts in the outstanding legs in the tension range shall conform to the following:

$$g \geq \sqrt{\frac{Lt}{12}}$$

Only high-strength bolts shall be used in simple connections.

Stress Induced by Out-of-Plane Displacement 11 — 7.2

Connections between members subjected to differential movements, both deflection and rotation, shall be detailed so that the movement will not result in unaccounted for cyclic stresses in short unstiffened gap regions. Attention shall be given to lateral gusset connections where relative deformation of the longitudinal members may result in undesirable out-of-plane movement of the gusset plate.

Attention shall be given to the floor beam, cross frame, and diaphragm connections. The transverse stiffeners serving as connection plates attached to the longitudinal girders shall preferably be welded or bolted to the tension and compression flanges.

Attention shall also be given to conditions due to shipping or handling which can result in cyclic stresses at restrained points.

Dynamic Response and Restraint **11 — 7.3**

End connections of long slender bracing members shall be detailed to minimize high cyclic stresses arising from vibrations.

Secondary Components **11 — 8**

General **11 — 8.1**

The secondary components shall be designed in accordance with Table 11 — 4.1 when they transmit forces due to passage of serviceability limit state Type I load.

Lateral Bracing, Diaphragms and Cross Frames **11 — 8.2**

Consideration shall be given to the forces developed in attachments to girders, especially gusset plates, as a result of the deformation of the girders.

Points of restraint where out-of-plane movements of the connected members create high cyclic stresses shall be avoided.

Dynamic Response **11 — 8.3**

Components, including lateral bracing susceptible to dynamic response due to highway live loads, hangers and other slender components subject to wind-induced vibrations, shall be designed so that they and their connections satisfy the allowable stress range provisions for over 2 000 000 cycles given in Table 11 — 4.1.

Aluminum Components — Allowable Stress Range in Fatigue **11 — 9**

Components, connections, mechanical fasteners and welds which are subjected to repeated variations of tensile stresses or to reversals of stress shall be proportioned so that the computed stress range does not exceed the allowable stress range, F_{sr}, given in Table 11 — 9.1 for the stress category specified in Clause 11 — 4.2.

Table **Allowable Stress Range in Fatigue of Aluminum** **11 — 9.1**

Stress Category	Allowable Stress Range F_{sr}, MPa			
	For 100 000 Cycles	For 500 000 Cycles	For 2 000 000 Cycles	Over 2 000 000 Cycles
a	120	95	75	65
b	110	85	65	50
c	97	70	52	40
d	76	50	35	25
e	55	35	25	15
f, w	40	27	20	12

Reinforcing Steel **11 — 10**

Stress Range in Reinforcement **11 — 10.1**

The stress range in straight reinforcement shall not exceed F_{sr} = 125 MPa.

Stress Range at Bends **11 — 10.2**

Bends shall not be permitted in Grade 350 reinforcement unless it satisfies the chemistry and bend requirements of Grade 400 reinforcement.

The stress range at bends meeting the requirements of Clause 8 — 9.1 shall not exceed 65 MPa.

Welded Reinforcement **11 — 10.3**

Tack welding shall not be permitted to primary reinforcement. For bars containing complete joint penetration groove welds conforming to the requirements of CSA Standard W186, *Welding of Reinforcing Bars in Reinforced Concrete Construction*, the stress range in the vicinity of welds shall not exceed 125 MPa.

For other types of welded splices, the stress range shall not exceed 65 MPa. Where reinforcement is spliced using fillet welded connections, the permissible stress range shall be as specified in Clauses 11 — 4.1 and 11 — 4.2.

Concrete Deck Slabs **11 — 10.4**

The requirements of Clause 11 — 10 need not be applied to the reinforcement perpendicular to traffic in concrete deck slabs designed in accordance with the method given in Clause 7 — 7.4, unless the reinforcement forms part of a longitudinal or transverse concrete or steel beam section.

Appendix — Fracture Control Plan — A11 — 1

Welding Requirements **A11 — 1.1**

Welding requirements of the CSA Standard W59, *Welded Steel Construction*, (metal-arc welding) apply as supplemented herein.

a) The Charpy V-notch impact energy requirements for weld metal in groove welds shall be in accordance with Table A11 — 1.

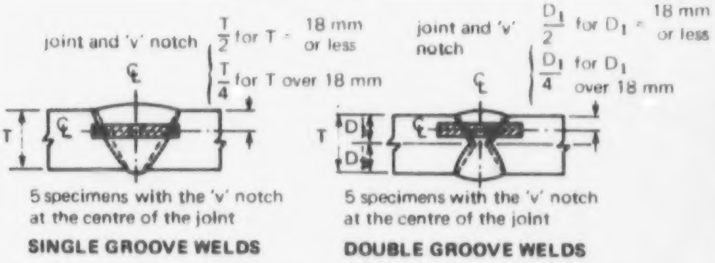
b) In groove welds connecting two different grades of steel, the electrodes used shall be those applicable to the steel having the lower ultimate tensile strength. The Charpy V-notch impact energy requirement for weld metal connecting the lower ultimate tensile strength steel shall apply.

Table **Charpy V-Notch (CVN) Test Requirements for Weld Metal** **A11 — 1**

Base Material Grade G40.21	Minimum Average Absorbed Energy	CVN Test Temperature	
		Service Temperature	
	(J)	≥ -40°C *	< -40°C †
260 WT	20	-30°C	-40°C
300 WT	20	-30°C	-40°C
350 WT, AT	27	-30°C	-40°C
400 WT, AT	27	-30°C	-40°C
480 WT, AT	27	-45°C	-45°C
700 QT	40	-45°C	-45°C

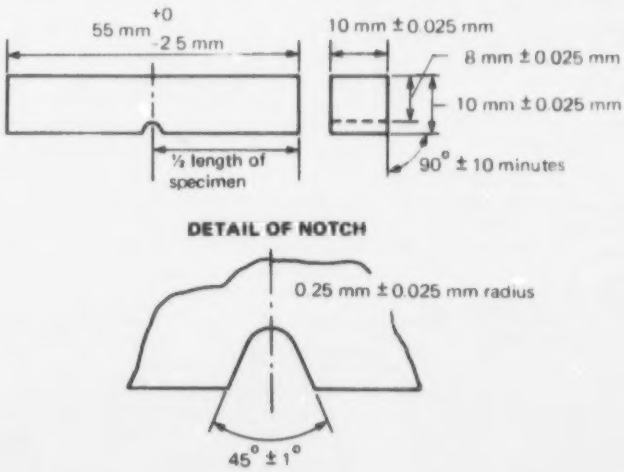
* i.e. warmer than -40°C † i.e. colder than -40°C

Figure **Impact Tests** **A11 — 1**


joint and 'v' notch
$\frac{T}{2}$ for T = 18 mm or less
$\frac{T}{4}$ for T over 18 mm
℄
T
5 specimens with the 'v' notch at the centre of the joint
SINGLE GROOVE WELDS
joint and 'v' notch
$\frac{D_1}{2}$ for D_1 = 18 mm or less
$\frac{D_1}{4}$ for D_1 over 18 mm
℄
T
D
D
5 specimens with the 'v' notch at the centre of the joint
DOUBLE GROOVE WELDS

(a) Location of Impact Specimens

Type of groove for illustration only


55 mm +0 -2.5 mm
½ length of specimen
10 mm ± 0.025 mm
8 mm ± 0.025 mm
10 mm ± 0.025 mm
90° ± 10 minutes
DETAIL OF NOTCH
0.25 mm ± 0.025 mm radius
45° ± 1°

(b) Impact Test Specimen

A11 — 1.2

a) Electrodes for shielded metal-arc welding shall be of the basic type. When the flux-cored arc welding process is used, all electrodes shall be of the controlled hydrogen (CH) designation. Submerged arc process electrode/flux combinations and all electrodes for any other process shall meet the applicable Charpy V-notch requirements of Clause A11 — 1.1.

Welding Procedures **A11 — 2**

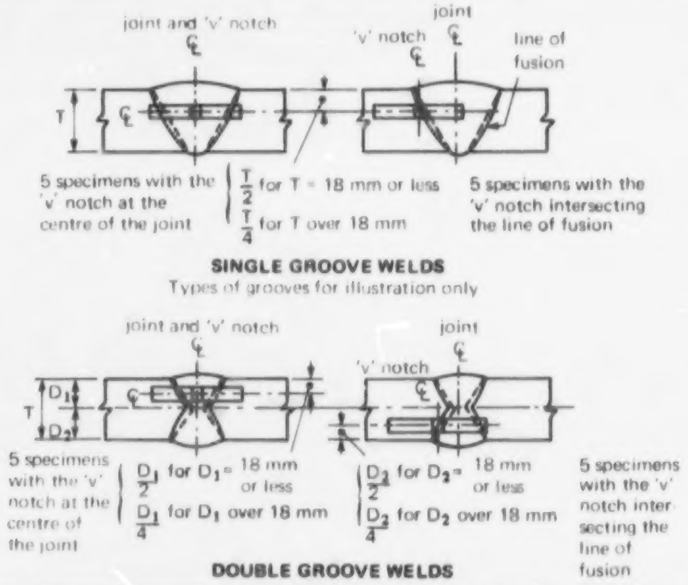
Approval

A11 — 2.1 Welding procedure specifications and related weld procedure data sheets shall be approved by the Canadian Welding Bureau.

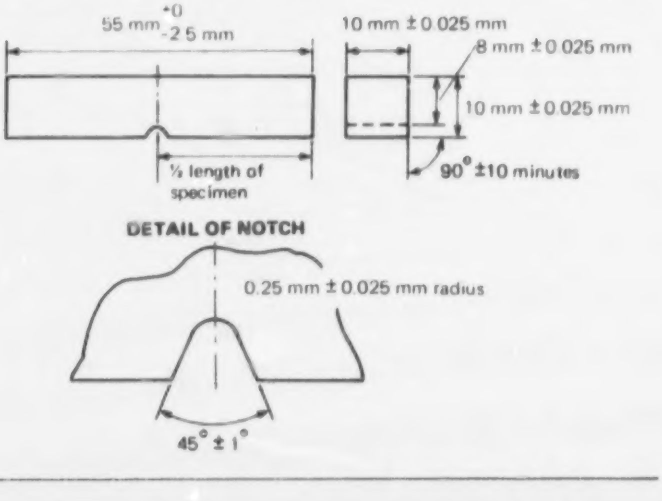
A11 — 2.1.1 Electrodes, electrodes and gas combinations, or electrodes and flux combinations shall be certified by the Canadian Welding Bureau to applicable CSA W48 standards or, in their absence, to AWS A5 specifications. The applicable requirements of Clause A11 — 1.1 shall be met.

A11 — 2.1.2 When the CSA W48 standards or the AWS A5 specifications require Charpy V-notch impact values at temperatures higher than those required by Clause A11 — 1.1 or when the welding consumables have not been previously certified by the Canadian Welding Bureau, such consumables shall be qualified by the Bureau in accordance

Figure **Impact Tests for Welds Connecting 700QT** **A11 — 2**


joint and 'v' notch
℄
joint
'v' notch
℄
℄
line of fusion
T
℄
5 specimens with the 'v' notch at the centre of the joint
$\frac{T}{2}$ for T = 18 mm or less
$\frac{T}{4}$ for T over 18 mm
5 specimens with the 'v' notch intersecting the line of fusion
SINGLE GROOVE WELDS
Types of grooves for illustration only
joint and 'v' notch
℄
joint
℄
'v' notch
℄
T
D_1
D_2
℄
5 specimens with the 'v' notch at the centre of the joint
$\frac{D_1}{2}$ for D_1 = 18 mm or less
$\frac{D_1}{4}$ for D_1 over 18 mm
$\frac{D_2}{2}$ for D_2 = 18 mm or less
$\frac{D_2}{4}$ for D_2 over 18 mm
5 specimens with the 'v' notch intersecting the line of fusion
DOUBLE GROOVE WELDS

(a) Location of Impact Specimens


55 mm $^{+0}_{-2.5}$ mm
10 mm ± 0.025 mm
8 mm ± 0.025 mm
10 mm ± 0.025 mm
½ length of specimen
90° ±10 minutes
DETAIL OF NOTCH
0.25 mm ± 0.025 mm radius
45° ± 1°

(b) Impact Test Specimen

Table **Preheat and Interpass Temperature** **A11 — 2**

Thickness (t) (mm)	G40.21 260 WT, 350 WT, 400 WT 480 WT, 300 WT, 350 AT 400 AT, and 480 AT	G40.21 700QT *
t ≤ 25	65°C	65°C
25 < t ≤ 40	120°C	120°C
t > 40	175°C	175°C

*Maximum preheat and interpass temperature shall not exceed recommendations of the steel manufacturer.

with the provisions of Clause 8 — 2.2.4 of CSA Standard W47.1 and shall include Charpy V-notch impact tests of weld material. The required Charpy V-notch properties specified in Clause A11 — 1.1 shall be established by qualification tests in accordance with the provisions of Figure A11 — 1. Only consumables of the manufacturer used in the qualification shall be permitted in production. Qualification shall be done for each lot or heat of the consumables.

A11 — 2.1.3 Welding procedures for 700QT material shall be certified by tests conducted by the Canadian Welding Bureau. The required Charpy V-notch properties specified in Clause A11 — 1.1 shall be established by qualification tests in accordance with the provisions of Figure A11 — 2. Only consumables of the manufacturer used in the qualification shall be permitted in production.

Repair Welding **A11 — 2.2** Except as specified in Clause A11 — 2.2.3(d)(1), repairs to base metal and to welded joints shall be documented. The documentation shall show all details specified in Clauses A11 — 2.2.5 and A11 — 2.2.6. Welding repair procedures shall be approved by the engineer in accordance with the provisions of Clauses A11 — 2.2.3 or A11 — 2.2.4, as applicable.

A11 — 2.2.1 Repair of base metal, by welding, at the producing mill shall be permitted only when the following conditions are met:

a) Preparation for repair welding shall include inspection to assure complete removal of the defect.
b) Repair procedures shall be approved by the engineer and shall be made using weld procedures and data sheets approved by the Canadian Welding Bureau and qualified welding operators.
c) If Charpy V-notch impact tests are required on the base metal, the welding procedure qualification tests shall also include impact tests of the weld. If an applicable and previously approved welding procedure is available but does not include the required Charpy V-notch impact tests, a new test assembly shall be prepared to provide the required impact tests in accordance with CSA Standard W47.1.
d) Quenched and tempered steels shall be repaired prior to heat treatment.

Notes for Figures A11 — 1 and A11 — 2

1/ Five specimens shall be cut out from test assembly at locations shown in Figure (a). Each specimen shall be finished as illustrated in Figure (b).
2/ Impact test values for weld metal specimens are as given in Clause A11 — 1.1.
3/ The impact specimens shall be tested in accordance with ASTM Standard E23, *Notched Bar Impact Testing of Metallic Materials*.
4/ Impact properties of the five specimens shall be obtained at the temperature indicated. The tolerance on the stated temperature shall be plus or minus 3°F.
5/ When computing the average value of the impact properties, the extreme lowest value and extreme highest value obtained with the five specimens shall be disregarded and the average of the remainder obtained.
6/ If the value for more than one of the three specimens is below the minimum average requirements, or if the value for one of the three specimens is below the minimum value permitted on one specimen, the test shall be repeated and the value of all the three specimens must equal or exceed the specified minimum average value. Such a retest shall be permitted only when the average value of the three specimens equals or exceeds the minimum value permitted on one specimen.
7/ Use in conjunction with Figure 2, Procedure Qualification Test Assembly, of CSA Standard W47.1, *Certification of Companies for Fusion Welding of Steel Structures*.

e) If the base metal is subjected to normalizing or quenching and tempering and the requirements of Clause A11 — 2.2.1(d) cannot be met, the weld procedure qualification test plates and the repaired base metal member shall be subjected to the same thermal heat treatment as the original base material.

f) Repair welds shall meet the quality requirements of Clause 12 of CSA Standard W59.

A11 — 2.2.2 Repair welding may be performed using any of the welding procedures employed in the fabrication of fracture critical members and primary tension members but the welding procedures shall be pre-approved by the Canadian Welding Bureau for that specific purpose. All repair welding shall be subject to non-destructive tests as provided herein.

A11 — 2.2.3 The contractor shall prepare repair procedures for non-critical repairs as defined in this clause and submit them to the engineer for pre-approval. These procedures shall apply to shop repair of discontinuities identified during in-process fabrication. Pre-approved repair procedures shall be employed after the inspector has verified that the discontinuity to be repaired is as described in the pre-approved procedure. Such repairs which may be pre-approved include the following:

a) the repair of welds requiring excavation of defects including laminations and cracks;

b) repair welding which consists solely of the deposition of additional weld metal to compensate for insufficient weld throat, overlap or undercut;

c) visually detected planar and laminar discontinuities as defined in Table 5 — 2 of CSA Standard W59, but not deeper than 25 mm, nor half of the thickness of the edge of the cut plate, whichever is less;
Such discontinuities shall not be within 300 mm of a tension groove weld. There shall also be no visible planar or laminar discontinuity on any prepared face of a tension groove joint prior to welding.

d) (1) gouges not more than 7 mm deep on otherwise satisfactory surfaces which may be repaired by machining and grinding;
   (2) occasional gouges exceeding 7 mm, but not more than 10 mm in depth on edges not to be welded, which may be repaired by welding.

For both types of repairs the procedural recommendations of Clause 5 — 3.2 of CSA Standard W59 shall be followed.

A11 — 2.2.4 Repair procedures beyond those described in Clause A11 — 2.2.3 are considered critical and shall be individually approved by the engineer before repair welding is begun. Critical repairs include the following:

a) repair of lamellar tearing;

b) repair of surface or internal defects in rolled products, except those meeting the requirements of Clause A11 — 2.2.3(d);

c) dimensional corrections requiring weld removal and rewelding;

d) any correction by welding to compensate for a fabrication error, such as improper cutting, punching, drilling, machining, fitting or assembly.

A11 — 2.2.5 Repair procedures in accordance with Clauses A11 — 2.2.3 and A11 — 2.3.4 shall include sketches or full size drawings as necessary to adequately describe the deficiency and the proposed method of repair. Critical repair procedures in accordance with Clause A11 — 2.2.4 shall include the location of the discontinuity.

A11 — 2.2.6 Repair procedures, except in cases meeting the requirements of Clause A11 — 2.2.3(b), shall include at least the following provisions and list steps in the order in which they should be performed.

a) Surfaces shall be cleaned and/or ground as necessary to aid visual and nondestructive tests to enable the contractor and inspector to identify and quantify the discontinuity(s).

b) The discontinuity shall be drawn as it appears from visual inspection and nondestructive testing (NDT).

c) Arc-air gouging, when required, shall be part of the approved welding procedure. The method and extent of excavation shall be completely described, giving the sequence of progressive excavations.

d) Magnetic particle inspection or other inspection method approved by the engineer shall be used to determine whether the discontinuity was removed as planned.

e) All air carbon-arc gouged and oxygen-cut surfaces that form a boundary for a repair weld shall be ground to form a smooth, bright surface. Oxygen gouging is not permitted.

f) All required run-off tabs and back-up bars shall be shown in detail.

g) Preheat and interpass temperature shall be in accordance with Table A11 — 2. Preheat and interpass temperatures shall be maintained without interruption until the repair is completed.

h) The repair procedures shall include the applicable welding procedure specification and the related data sheet. If both were approved by the Canadian Welding Bureau prior to fabrication, they need not be qualified by test for the specific method of repair unless a change in essential variables has been made or unless otherwise required by the engineer.

i) If the geometry of the repair joint or of the excavation is similar to the geometry of a prequalified joint preparation as defined in CSA Standard W59, and permits good access to all portions of such joints or excavations during the proposed sequence of welding, it shall not require qualification by test unless required by the engineer.

j) Peening, when required, shall be completely described and shall be approved. Peening equipment shall not contaminate the joint.

k) Postheat shall be employed and shall continue without interruption from the completion of repair welding to the end of the minimum specified postheat period. Postheat of the repair area shall be between 200°C and 260°C and shall be for a period of one hour minimum for each 25 mm of weld thickness, or for two hours, whichever is less.

l) Faces of repair shall be ground flush with the plate or blended to the same contour and throat dimension as the remaining sound weld.

m) If stress relief heat treatment is required, it shall be completely described. Final acceptance nondestructive testing (NDT) shall be performed after stress relief is complete.

n) Repairs of groove welds in fracture critical members shall be examined by ultrasonic testing (UT) and radiographic testing (RT). Repairs to groove welds in primary tension members shall be examined by UT or RT. Fillet weld repairs shall be examined by magnetic particle testing (MT). RT shall conform to Clause 8 — 1 of CSA Standard W59 and may be performed as soon as the weldment has cooled to ambient temperature. MT and UT shall conform to Clause 8 — 2 and Clause 7 — 4.4 of CSA Standard W59, respectively. Final acceptance testing by magnetic particle and ultrasonic methods shall not be performed until the steel weldments have been cooled to ambient temperature for at least the elapsed time indicated in Table A11 — 3.

Table **Minimum Elapsed Time for Acceptance Testing** **A11 — 3**

Plate Thickness (mm)	Magnetic Particle for Fillet Welds	Ultrasonic Examination of Groove Welds
t ≤ 50	24 h	24 h
t > 50	24 h	48 h

A11 — 2.2.7 All repair welding and nondestructive testing shall be performed as described in the approved repair procedure.

A11 — 2.2.8 Approved critical repair procedures shall be retained as part of the project records.

Nondestructive Testing of Fracture Critical Members

A11 — 3 Quality control (QC) shall be the responsibility of the fabricator/erector, and quality assurance (QA) the prerogative of the engineer.

A11 — 3.1 Repair welds shall be examined in accordance with Clause A11 — 2.2.6(n).

A11 — 3.2 Cobalt 60 may be used as a radiograph source in quality control (QC) only when the steel being tested exceeds 75 mm in thickness.

A11 — 3.3 The fabricator/erector shall maintain documentation of all visual and nondestructive testing for timely review and confirmation by the engineer. Documentation shall be turned over to the engineer upon completion of the project.

12 Soil–Steel Structures

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Reference Publications

This section makes references to the following publications

ASTM Standards	D 698-70	*Standard Test Methods for Moisture-Density Relations of Soils using 2.5 kg Rammer and 304.8 mm Drop*
	D 2487-69	*Standard Test Method for Classification of Soils for Engineering Purposes*
CSA Standard	CAN3-G401-M81	*Corrugated Steel Pipe Products*

Scope **12-1**

This section specifies minimum requirements for the analysis and design of soil-steel structures. Appendix A12-2 provides tables of permitted design thickness of conduit walls with spans up to 8 m having a 152 mm x 51 mm corrugation profile and subjected to loading specified in Section 2. This section also specifies construction procedures and requirements for construction supervision.
Certain special techniques which may be beneficial to soil-steel interaction are permitted and are included in this section. The use of other techniques is subject to approval.

Definitions **12-2**

Arch — a soil-steel structure in which the conduit wall is not continuous around the perimeter of the bridge opening, and in which the conduit wall ends are supported on concrete footings.
Arching — the effect produced by transfer of pressure between adjoining soil masses which move relative to each other. Positive or active arching is that which results in the transfer of loads away from the conduit; negative or passive arching produces the opposite effect.
Backfill — the envelope of engineered soil, excluding the bedding, placed around the conduit in a controlled manner.
Bedding — the prepared portion of the engineered soil on which the conduit invert is placed.
Bevelled End — termination of the steel wall of a conduit, cut at a plane inclined to the horizontal.
Camber — a measure of adjustment required in the longitudinal profile of the bedding in order to compensate for post-construction settlement.
Compaction — the process of soil densification, at a specified moisture content, due to application of loads through rolling, kneading, tamping, rodding, or vibratory actions of mechanical or manual equipment.
Conduit — a general term for the bridge opening in a soil-steel structure.
Crown — the highest point on a transverse section of a conduit.
Depth of Cover — the vertical distance between the profile grade, including pavement, and the crown.
Engineered Soil — a selected soil of known properties placed around a conduit in a prescribed manner.
Flexibility Factor — a measure of conduit wall flexibility for handling and installation purposes.
Foundation — soil or rock materials underlying the engineered soil.
Free-Field Overburden Pressure — the vertical earth pressure at a level in a semi-infinite mass due to the load of earth and other materials above that level.
Haunch — a portion of conduit cross-section between the spring line and the top of the bedding or footing.
Horizontally-Elliptical Pipe — an elliptical conduit with major diameter horizontal and not less than 1.10 times the minor diameter.
Longitudinal Direction — the direction of the conduit axis which is parallel to the locus of the crown of a conduit.
Modulus of Soil Reaction — ratio of the intensity of radial contact pressure to the average radial strain in the soil.
Pipe-Arch — a conduit consisting of an arched upper portion which is structurally continuous with an invert whose radius of curvature is greater than that of the upper portion.
Profile Grade — the top of the finished granular base at the centre line of the highway.
Re-Entrant Arch — an arch with haunches.
Rise — the maximum vertical clearance inside a conduit at a given transverse section.
Round Pipe — a circular or elliptical conduit with a major diameter not exceeding 1.10 times the minor diameter.
Seam — a joint between two structural steel plates formed by overlapping and bolting them together.
Shoulder — the portion of the conduit between the crown and the spring line.
Soil Liquefaction — loss of strength of a soil resulting from the combined effects of vibrations and hydraulic forces, thereby causing the material to flow.
Soil-Steel Structure — a bridge, comprised of structural steel plates and engineered soil, designed and constructed to induce a beneficial interaction of the two materials.
Span — the maximum horizontal clearance inside a conduit at a given transverse section.
Spring Lines — the loci of horizontal extremities of transverse sections of the conduit.

Standard Proctor Density — the optimum unit weight of a soil determined in accordance with ASTM Standard D698-70.

Thrust — the circumferential compressive force in the conduit wall, per unit length of conduit.

Transverse Direction — the direction perpendicular to the longitudinal direction.

Transverse Section — a section in the vertical plane normal to the horizontal projection of the longitudinal direction.

Vertically-Elliptical Pipe — an elliptical conduit with major diameter vertical and not less than 1.10 times the minor diameter.

Notation

12—3

A	cross-sectional area of corrugated steel conduit wall per unit length, in the longitudinal direction mm²/mm.
b	height of bevel cut, m.
D	equivalent diameter, metres $= \frac{1}{\pi}$ (perimeter of the conduit in metres).
D_h	span, m.
D_v	rise, m.
d	depth of corrugation, mm.
E	modulus of elasticity of steel, MPa.
E'	modulus of soil reaction, MPa.
$\bar{E}$	modified modulus of soil reaction, MPa.
F	flexibility factor, mm/N.
f_b	factored buckling stress in the conduit wall, MPa.
f_y	yield stress of the conduit wall, MPa.
g	acceleration due to gravity, m/s².
H	depth of cover $= \sum_{i=1}^{n} h_i$, m.
h_i	thickness of the ith layer above the crown, m.
I	dynamic load allowance expressed as a fraction of live load.
I_s	second moment of cross-sectional area, A, about its longitudinal axis, mm⁴/mm.
i	material layer number above the crown.
K	a factor representing the relative stiffness of the conduit with respect to the adjacent soil.
ℓ_t	a length defined in Clause 12—5.5.1.3, equal to (1.45 + 2H), in metres.
m_f	modification factor for multi-lane loading.
n	number of different material layers above the crown.
R	radius of curvature of the conduit wall, taken inside the conduit, at a transverse section, mm.
$\bar{R}$	an equivalent radius $= \frac{r}{K}\left[\frac{6E\rho}{f_y}\right]^{0.5}$, mm.
R_t	radius of conduit at crown, mm.
r	radius of gyration of corrugation profile $= [I_s/A]^{0.5}$, mm.
S_s	ultimate longitudinal seam strength, per unit length, kN/m.
T	thrust in conduit wall per unit length, kN/m.
T_D, T_L	maximum thrust per unit length due to dead and live loads, respectively, kN/m.
α_D	dead load factor for soil, specified in Section 2.
α_L	live load factor, specified in Section 2.
γ_{si}	unit of weight of soil or other material in the ith layer, kN/m³.
δ_{crown}	vertical deflection at crown, mm.
θ	skew angle of the conduit as defined in Table 12—7.3.
λ	a factor governing K.
μ_1, μ_2	conduit shape factors.
σ	stress in a conduit wall = T/A, MPa.
σ_L	equivalent uniformly-distributed pressure at the crown due to the dispersed live load, kPa.
σ_{vo}	free field overburden pressure, kPa.
ρ	reduction factor for buckling stress in conduit walls.
ϕ	performance factor.

Hydraulic Design

12—4

The hydraulic design of a soil-steel structure intended mainly to convey water shall be in accordance with the provisions of Section 1.

Structural Design 12—5

Limit States 12—5.1

The following ultimate limit states shall be considered in design:

a) Buckling of Conduit Walls: in accordance with Clause 12—5.5.2;
b) Failure of Seams: in accordance with Clause 12—5.5.3.

Performance Factors 12—5.2

The following values of the performance factor ϕ shall be used:

a) for conduit walls to be used in conjunction with Clause 12—5.5.2:

$\phi = 0.80$, for structures with R_t less than or equal to 4000 mm;

$\phi = 0.75$, for structures with R_t greater than 4000 mm;

b) for seams, to be used in conjunction with Clause 12—5.5.3, ϕ shall be 0.70.

Geotechnical Considerations 12—5.3

12—5.3.1 Foundation Investigation

The feasibility of constructing a soil-steel structure and its approaches shall be established by a foundation investigation of the site.

This requirement may be waived when knowledge of local subsurface conditions precludes the possibility of instability of approach fills and cuts during and after construction, and predicts settlements reliably.

12—5.3.2 Selection of Materials

Materials selected for the engineered soil shall be non-cohesive and capable of being compacted to densities assumed in design. Other types of soils may be used provided that their engineering characteristics are predetermined and the design is based upon these characteristics.

12—5.3.3 Soil Properties

The design of soil-steel structures shall be based on recommendations provided in the appropriate geotechnical report. If specific values of soil properties are not available, the values provided in Appendix A12—1 shall be used for design.

12—5.3.4 Camber

The bedding grade and initial conduit invert elevations shall be designed to ensure the achievement of the final invert profile gradient after anticipated foundation settlements have occurred.

12—5.3.5 Radius of Curvature

The radius of curvature of the conduit wall at any location away from the crown shall not be less than 1/5th of that at the crown, unless approved.

12—5.3.6 Arch Footings

Footings for arches shall be designed in accordance with the provisions of Section 6. Scour protection shall be provided in accordance with the provisions of Section 1.

Design Table 12—5.4

The structural design table for selection of conduit wall thickness provided in Appendix A12—2 may be used in the design of round pipes up to a span of 8 m, when the design highway loading is applied at the profile grade of the embankment.

Design Criteria 12—5.5

12—5.5.1 Thrust

12—5.5.1.1 General

The thrust, T, in the conduit wall at the ultimate limit state due to live loads and dead loads only shall be calculated according to the following equation, which incorporates load combination No. 1 for the ultimate limit states of Table 2—5.1(a).

$$T = \alpha_D T_D + \alpha_L T_L$$

where T_D and T_L are calculated in accordance with Clauses 12—5.5.1.2 and 12—5.5.1.3, respectively. Alternatively, T_D and T_L may be obtained by a plane frame method, in which the structure is analysed as a frame supported on discrete springs, or by the finite element method. Details of such analyses shall be subject to approval.

12—5.5.1.2 Dead Loads

The thrust acting on the conduit walls of soil-steel structures due to the overburden loading shall be calculated as:

$$T_D = \frac{\mu_1 \sigma_{vo} R_t}{1000}$$

where: $\sigma_{vo} = \sum_{i=1}^{n} \gamma_{si} h_i$

and the value of μ_1 is determined from Figure 12—5.5.1.2.

12—5.5.1.3 Live Loads

The thrust, T_L, shall be assumed to be uniform around the conduit, and its value shall be obtained by:

$$T_L = 0.5\,(\text{lesser of } D_h \text{ and } \ell_t)\, \sigma_L\, m_f (1 + I)$$

where σ_L, the equivalent uniformly-distributed load at the crown, is obtained by positioning the 280 kN tandem of the OHBDC truck centrally above the crown at embankment level; distributing the load through fill at a slope of one vertically to one horizontally in the transverse direction of the conduit and a slope of two vertically to one horizontally in the longitudinal direction; and assuming that the dispersed load at crown level is uniformly distributed over a rectangular area that is subtended by the extremities of individual dispersed areas. In this formula, ℓ_t, being the length of the dispersed load along the conduit span, is equal to (1.45 + 2H) metres. The dynamic load allowance, I, shall be obtained from Clause 2—4.3.2, and the modification factor for multi-lane loading, m_f, from Clause 2—4.3.1.9. Axles other than the abovementioned two 140 kN axles of the OHBDC truck and OHBD lane loads shall not be considered in the design of soil-steel structures.

12—5.5.2 Strength Requirements

At the ultimate limit state the stress, $\sigma = \frac{T}{A}$ shall not exceed the factored bucking stress given either by:

$$f_b = \phi\left\{f_y - \left[\frac{f_y^2}{12E}\left(\frac{KR}{r}\right)^2\frac{1}{\rho}\right]\right\} \text{ for } R \le \bar{R}$$

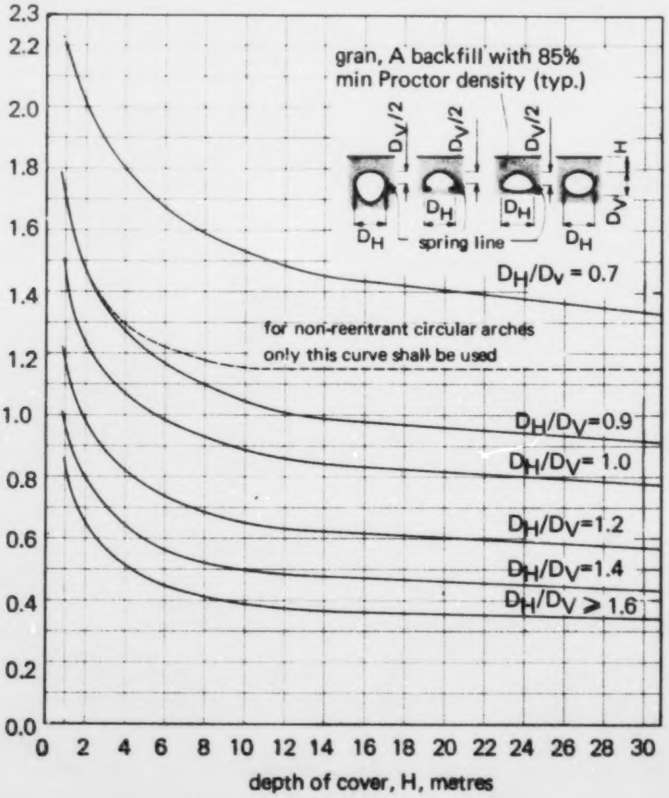
or: $$f_b = \phi\frac{3E\rho}{\left(\frac{KR}{r}\right)^2} \text{ for } R > \bar{R}$$

where: ϕ is taken from Clause 12—5.2

and: $$\bar{R} = \left[\frac{r}{K}\frac{6E\rho}{f_y}\right]^{0.5}$$

and: $$K = \lambda\left[\frac{EI_s}{\bar{E}R^3}\right]^{0.25} \le 0.5$$

Figure **Values of μ_1** **12—5.5.1.2**



where: λ = 1.22 for conduit walls of circular arches with rise-to-span ratios of less than 0.4, and for side and bottom portions of conduit walls of all other structures;

or: $$\lambda = 1.22\left[1.0 + 2\left(\frac{EI_s}{\bar{E}R_t^3}\right)^{0.25}\right]$$

for upper portions of conduit walls in all structures except circular arches with rise-to-span ratios of less than 0.4.

$$\bar{E} = E'\left[1 - \left(\frac{R_t}{R_t + 1000H}\right)^2\right]$$

$$\rho = 1.0 \text{ for } \left[\frac{1000H}{2R_t}\right] \geq 1.0$$

or: $$\left[\frac{1000H}{2R_t}\right]^{0.50} \quad \text{for} \quad \left[\frac{1000H}{2R_t}\right] < 1.0$$

12—5.5.3 **Seam Strength**

The factored longitudinal seam strength, ϕS_s, of conduit wall shall not be less than the thrust due to factored loads at the ultimate limit state evaluated in accordance with Clause 12—5.5.1.1, and using $\phi = 0.7$. The value of S_s shall be evaluated experimentally or shall be obtained from Figure 12—5.5.3.

12—5.5.4 **Handling and Installation Strength**

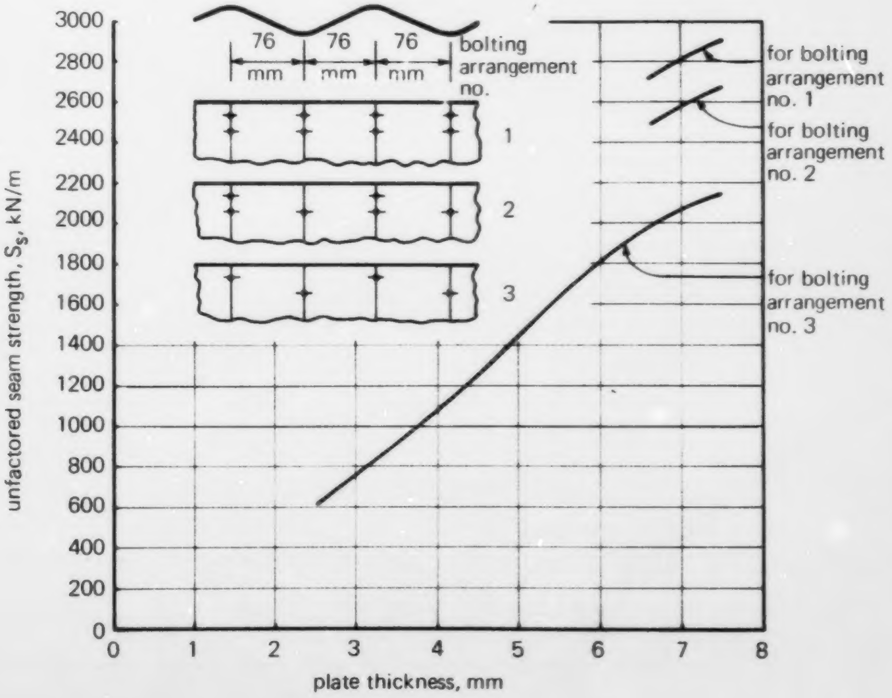
The conduit wall flexibility factor F required for handling and installation, shall not exceed 0.114 mm/N, unless experience justifies the use of a higher value.

The value of F shall be calculated as follows:

$$F = \frac{(D^2 \times 10^6)}{EI_s}$$

When F exceeds 0.114, the plans shall specify fabrication and construction techniques.

Figure **Longitudinal Seam Strengths of Bolted Plates With Diameter Bolts** **12—5.5.3**



12—5.5.5 **Maximum Difference in Plate Thickness**

The maximum difference in the thickness of the plates meeting at a longitudinal seam shall not exceed 1 mm if one of the plates has a thickness of less than 3.1 mm; nor exceed 1.5 mm if one of the plates has a thickness between 3.1 and 3.5 mm. When each plate thickness exceeds 3.5 mm, there is no restriction on the difference in plate thicknesses at a longitudinal seam.

Durability **12—6**

Steel plates and components shall satisfy material, fabrication, and hot-dip galvanizing requirements of CSA CAN3-G401-M81.

Special Design Considerations **12—7**

Minimum Depth of Cover **12—7.1**

The minimum depth of cover over the conduit shall be the larger of:

$$\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{0.5} \text{ and } 0.6\left(\frac{D_h}{D_v}\right)^{2} \text{ metres, with a minimum of 0.6 m.}$$

This requirement may be waived if the design method is approved.

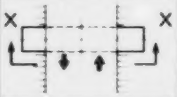
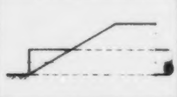
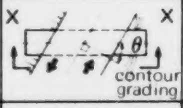
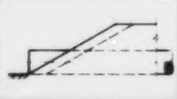
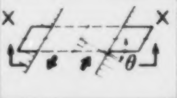
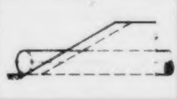
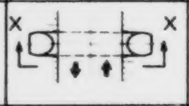
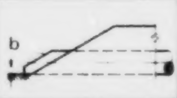
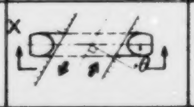
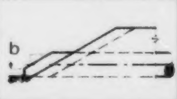
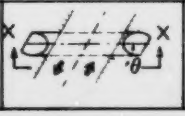
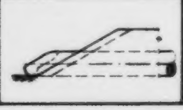
Multiple Structures **12—7.2**

In a multi-conduit soil-steel structure installation, the clear spacing between any two adjacent conduits shall not be less than 1.0 m or one-tenth of the larger of the two adjacent spans, whichever is greater. This requirement may be waived if it can be shown by an approved method that the stresses in the walls of each conduit, including the stresses due to their interaction, do not exceed those for which they were individually designed.

Cut Ends **12—7.3**

Requirements for various combinations of cut ends and conduit installations shall be as specified in Table 12—7.3.

Table **Requirements for Cut Ends** **12—7.3**

		Typical View of Installation		
		Plan	Section XX	
1	square end with roadway parallel to transverse direction			
2	square end with roadway skew to transverse direction			θ shall be less than 40°. For $\theta > 20°$, the earth pressure imbalance shall be accommodated either by structural reinforcement of conduit wall or by contour grading of the embankment slope.
3	skew end			As for No. 2.
4	square bevel with roadway parallel to transverse direction			b shall not be less than (rise/8). The ends shall be treated as a retaining structure and shall be designed in accordance with Section 6.
5	square bevel with roadway skew to transverse direction			As for Nos. 3 and 4.
6	skew bevel			Requires approval.

Structures in Earthquake Zones 12—7.4

In earthquake zones, as defined in Section 2, soil-steel structures and associated embankments shall be designed to satisfy the following additional design provisions.

a) Earth embankments situated on sensitive clays, or earth cuts within sensitive clays, shall be analysed for instability using a horizontal acceleration of 0.10 g.

b) Bedding and backfill material, as well as any material used below the bedding in subexcavations, shall be selected and placed to prevent liquefaction.

Construction and Installation 12—8

General 12—8.1

The plans shall specify appropriate construction procedures and controls to ensure conformity with design.

Deformation During Backfilling 12—8.2

For elliptical and round conduits, the maximum value of upward crown deflection during the backfilling shall not exceed δ_{crown}, where:

$$\delta_{crown} = 1000\mu_2 \frac{D_h^2}{d}$$

and the values of the shape factor, μ_2, are obtained from Figure 12—8.2.

For other conduit shapes, the upward crown deflection shall not exceed 5% of the rise unless approved otherwise.

Foundation 12—8.3

Foundations for soil-steel structures, where possible, shall have uniform settlement properties throughout the extent of the conduit and its backfill. Provisions of camber, in accordance with Clause 12—5.3.4, shall be met by cutting the foundation or shaping the bedding to the required grade.

Material unsuitable for foundation purposes shall be removed and replaced with acceptable materials as recommended in the geotechnical report or as found necessary during construction.

Bedding 12—8.4

The bedding shall be preshaped in the transverse direction to accommodate the conduit invert curvature or shall consist of a flat bed to which fill is added under haunches after the conduit has been set in place. A 200 mm layer of that portion of the bedding which is in direct contact with the conduit wall shall be left uncompacted.

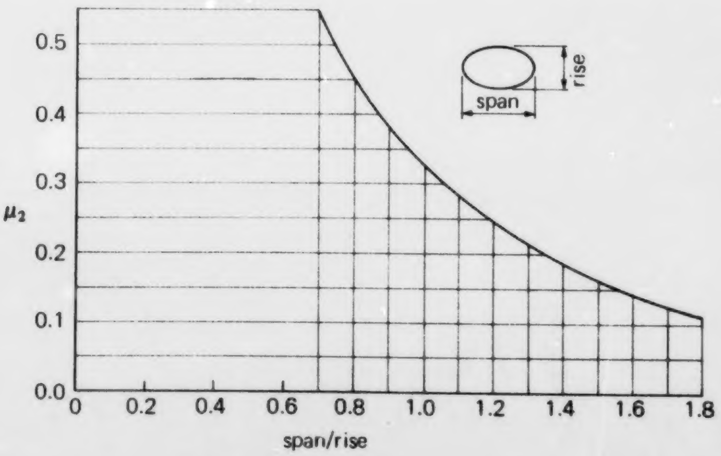
Assembly and Erection 12—8.5

The assembling and erection of the conduit wall shall be such that no permanent set results in any portion of the wall. In bolted structures the initial torque on the bolts shall be between 200 and 340 Nm. To ensure conformity to these torquing values, the following minimum percentages of bolts shall be tested after completion of erection of the conduit walls and before the backfilling operation:

a) circumferential seams — 5% of bolts in each circumferential seam;

b) longitudinal seams: — 5% of bolts in each longitudinal seam in each plate.

igure **μ_2 for Conduits of Elliptical and Round Shapes** **12—8.2**



The test bolts shall be selected in a random manner, and the installation shall be considered acceptable if the above torque requirements are met in at least 90% of the bolts tested.

Struts may be used to support the conduit wall during the backfilling but shall not be allowed to restrict the free downward movement of the crown once the backfill has reached the crown level. Struts shall be removed after completion of the backfill.

Backfill 12—8.6

Backfill material, which shall be boulder free, shall be placed and compacted in layers not exceeding 300 mm of compacted thickness, with each layer compacted to the required density prior to the addition of the next layer, and on both sides of the conduit such that the difference in the levels of backfills on the two sides at any transverse section does not exceed 600 mm.

Stones exceeding 80 mm in size shall not be allowed within 300 mm of the conduit walls. Heavy vibratory equipment shall not be allowed so near to the conduit wall that excessive distortion in the wall may occur.

The backfill shall extend transversely at least half the span on each side beyond the springing lines for the conduit, and vertically up to the minimum depth of cover required in accordance with Clause 12—7.1, above the crown.

Headwalls and Appurtenances 12—8.7

Soil-steel structures designed for hydraulic service shall be provided with headwalls and cut-off appurtenances when required by the provisions of Clause 1—7.

Special Techniques 12—9

The following special techniques may be employed for improving the soil-steel interaction and/or structural integrity:

a) longitudinal beams connected to the conduit wall at the soil-steel interface in shoulder areas;
b) a reinforced or prestressed concrete relieving slab above the crown;
c) transverse reinforcing ribs of steel, structurally connected to the conduit wall;
d) squeeze blocks of treated wood incorporated in footings of an arch.

Notwithstanding the use of any of the above special techniques, all requirements of this section shall still be met unless otherwise approved.

The use of special techniques other than those listed above requires approval.

Site Supervision and Construction Control 12—10

The plans shall require that the engineer designated by the owner as being responsible for inspection or supervision shall be experienced in the construction of soil-steel structures. The supervision of construction by the engineer shall be provided on the following basis.

a) For soil-steel structures of spans between 3 and 6 m, the work shall be inspected by an engineer or his representative at the completion of each of the following stages: foundation; bedding; erection; and placement of backfill under haunches, up to spring line, up to crown, and up to level of minimum cover.
b) For soil-steel structures with spans greater than 6 m but less than or equal to 8 m, the inspection by the engineer shall be as in (a) above. In addition, daily inspection shall be made during the backfilling operations until minimum cover is attained.
c) For soil-steel structures with spans greater than 8 m, or those in which special techniques are used in accordance with Clause 12—9, continuous inspection by the engineer or his representative is required during all stages of operations involving foundations, bedding, erection and backfilling as in (b) above.

Appendix A12

Soil Properties A12—1

In the absence of actual test data or geotechnical recommendations, the design values of E' for the various soils classified according to Table A12—1(a) shall be obtained from Table 12—1(b).

Design Table A12—2

For backfills consisting of soil group No. 1, in accordance with Table A12—1(a), compacted to Proctor densities of between 85 and 95%, the conduit wall thickness for round pipes under various depths of cover may be selected from Table A12—2. The wall thicknesses in the table are based upon compliance with Clauses 12—5.5.1.1., 12—5.5.1.2, 12—5.5.1.3, 12—5.5.2, and 12—5.5.5.

Table **Soil Classification for E'** **A12 — 1(a)**

Soil Group Number	Grain Size	Soil Types Included	Unified Soil Classification Symbol*
I	Coarse	Well graded gravel or sandy gravel.	GW
		Poorly graded gravel or sandy gravel.	GP
		Well graded sand or gravelly sand.	SW
		Poorly graded sand or gravelly sand.	SP
		Granular 'A' soil**	GW, SW
		Granular 'B' soil**	SW, SP
II	Medium	Clayey gravel or clayey-sandy gravel.	GC
		Clayey sand or clayey gravelly sand.	SC
		Silty sand or silty gravelly sand.	SM
III	Fine	Silts, sandy silts or plastic silts.	ML
		Lean clays, clayey silts.	CL

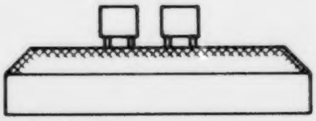
* According to ASTM D 2487-69
** According to MTC form 1010.

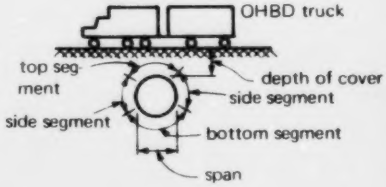
Table **Values of E' for Various Soils** **A12 — 1(b)**

Soil Group Number*	State of Compaction	Standard Proctor Density**	Modulus of Soil Reaction, E', MPa
I	Loose	Less than 85%	6.5
	Medium	Between 85% and 95%	13.0
	Dense	Greater than 95%	20.0
II	Loose	Less than 85%	4.0
	Medium	Between 85% and 95%	9.0
	Dense	Greater than 95%	12.0
III	Loose	Less than 85%	1.4
	Medium	Between 85% and 95%	1.7
	Dense	Greater than 95%	2.0

* From Table A12 — 1(a)
** According to ASTM D698-70

Table **Design Conduit Wall Thickness for Round Pipes** **A12 — 2**





Span in Metres (feet equivalent in parentheses)	Segment	0.6	0.8	1.0	1.5	2.0	2.5	3.0	4.0	5.0	6.0	8.0	10.0	12.5	15.0	17.5	20.0	25.0	30.0
		Design Thickness in Millimetres for Conduit Walls with 152 mm × 51 mm Corrugation Profile Depth of Cover in Metres																	
3.35 (11')	Top	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	6.0	7.0
	Side and Bottom	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	4.0	5.0	6.0
3.66 (12')	Top	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	5.0	6.0	7.0
	Side and Bottom	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	4.0	5.0	5.0	6.0

Table **Design Conduit Wall Thickness for Round Pipes (Cont'd)** **A12—2**

Span in Metres (feet equivalent in parentheses)		Design Thickness in Millimetres for Conduit Walls with 152 mm × 51 mm Corrugation Profile Depth of Cover in Metres																	
	Segment	0.6	0.8	1.0	1.5	2.0	2.5	3.0	4.0	5.0	6.0	8.0	10.0	12.5	15.0	17.5	20.0	25.0	30.0
3.96 (13′)	Top	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	6.0	7.0	
	Side and Bottom	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	5.0	6.0	
4.27 (14′)	Top		3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	5.0	6.0	6.0		
	Side and Bottom		3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	5.0		
4.57 (15′)	Top		3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	6.0	7.0		
	Side and Bottom		3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	5.0	6.0		
4.88 (16′)	Top			3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	5.0	5.0	6.0	7.0		
	Side and Bottom			3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0	5.0	5.0	6.0		
5.18 (17′)	Top			4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	5.0	6.0	7.0			
	Side and Bottom			4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	5.0	5.0	6.0			
5.49 (18′)	Top			4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	5.0	6.0				
	Side and Bottom			4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	5.0	5.0				
5.79 (19′)	Top			5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	6.0	7.0				
	Side and Bottom			5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	6.0				
6.10 (20′)	Top				5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	6.0					
	Side and Bottom				5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0					
6.40 (21′)	Top				5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	6.0					
	Side and Bottom				5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0					
6.71 (22′)	Top				6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	7.0					
	Side and Bottom				6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0					
7.01 (23′)	Top				6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0						
	Side and Bottom				6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0						
7.32 (24′)	Top				7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0						
	Side and Bottom				7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0						
7.62 (25′)	Top				7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0						
	Side and Bottom				7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0	7.0						

13 Wood Structures

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Reference Publications

Section 13 makes reference to the following publications:

CSA Standards

B111-1974	*Wire Nails, Spikes and Staples*
G30.5-1972(R1979)	*Welded Steel Wire Fabric for Concrete Reinforcement*
G164-1965(R1972)	*Hot-Dip Galvanizing of Irregularly-Shaped Articles*
G279-M1982	*Steel for Prestressed Concrete Tendons*
CAN3-056-M79	*Round Wood Piles*
080-1974 080-S4-1980	*Wood Preservation*
CAN3-086-M80	*Code for Engineering Design in Wood*
0122-M1980	*Structural Glued-Laminated Timber*
0141-1970	*Softwood Lumber*
0177-1977	*Qualification Code for Manufacturers of Structural Glued-Laminated Timber*

National Lumber Grade Authority

- "Standard Grading Rules for Canadian Lumber," NLGA, December, 1980

ASTM Standard

D245-74	*Establishing Structural Grades and Related Allowable Properties for Visually-Graded Lumber*

Scope **13—1**

The provisions of this section apply to all structural wood elements and their fastenings for bridges carrying highway or pedestrian traffic.

Definitions **13—2**

Beam and Stringer — a grading term referring to sawn wood having a mininimum thickness of 114 mm and a width at least 51 mm greater, stress-graded for use in bending with the load applied to the narrow face.

Bearing Block — a short wood block with grain parallel to the applied prestress force used to distribute prestress forces in a steel distribution bulkhead of a prestressed wood bridge using an external prestressing system.

Butt Joint — a term referring to the discontinuities in a laminated wood deck where the ends of two laminations meet.

Crib — a configuration of horizontal members with alternate layers usually perpendicular to one another, connected together to form a closed box.

Dimension Lumber — sawn wood with a thickness of 38 to 89 mm.

Direct Bearing Area — the area of outside lamination over which the prestressing is assumed to be applied.

Direct Bearing Pressure — the average pressure which is assumed to be applied to the direct bearing area due to the prestressing force.

Distributed Normal Pressure — the uniform pressure on a prestressed wood deck normal to the direction of the laminations and at some distance away from the distribution bulkhead.

Distribution Bulkhead — a continuous steel section, used along each side of a prestressed wood deck, which distributes the prestress forces.

Drift Pin — a steel pin used to connect wood members.

Duration of Load — a period of continuous application of a specified load or the summation of the time periods of intermittent applications of the same load.

External Prestressing System — a method of transversely prestressing a longitudinally-laminated wood deck using two bars at each station, one above the deck and one below.

Framed Bent — usually a single line of wood columns suitably braced.

Glued-Laminated Timber — a structural wood component produced by gluing together a number of laminations having their grain essentially parallel.

Grade — the designation of the size or quality of a piece of wood visually stress-graded in accordance with the NLGA Standard Grading Rules for Canadian Lumber.

Header — a horizontal member of a crib whose longitudinal axis runs perpendicular to the long side of the crib and which provides anchorage to the stretchers.

Incising — the process of inserting and withdrawing a series of closely-spaced knife points into the surface of wood prior to pressure preservative treatment.

Internal Prestressing System — a method of transversely prestressing a longitudinally-laminated wood deck using a single bar at each station, situated at the neutral axis of the wood deck.

Joist — a grading term referring to sawn wood 38 to 89 mm in thickness having a width of at least 102 mm, intended to be loaded on its narrow face.

Lamination — a term referring to dimension lumber used in a laminated wood deck or beam.

Laminated Wood Deck — a deck composed of dimension lumber, with the widths oriented in the vertical direction, joined together to form a continuous wood slab.

Load-Sharing System — a construction composed of three or more essentially parallel members so arranged or connected that they mutually support the load; such that, with the theoretical removal of at least one member, the system retains the capacity to support the load.

Longitudinally-Laminated Deck — a laminated wood deck where the length of the laminations is oriented in the direction of the span of the bridge.

Longitudinal Shear — the vertical shear in the direction of the span of the member.

Nailed-Laminated Deck — a laminated wood deck joined together only by the successive nailing of each lamination to the preceding one.

Pile Bent — usually a single line of free-standing piles, suitably braced and connected to form a pier.

Plank — a grading term referring to sawn wood 38 to 89 mm in thickness having a width of at least 114 mm, intended to be loaded on its wide face.

Post — a grading term referring to sawn wood having a minimum thickness of 114 mm and a width not more than 51 mm greater than the thickness; graded for use as a column.

Preservative Treatment — unless otherwise noted, impregnation under pressure with a wood preservative, in accordance with the requirements of CSA Standard 080.

Prestressed Wood Deck — a longitudinally-laminated wood deck which is transversely prestressed perpendicular to the deck laminations using high-strength steel bars.

Prestress Forces — the forces applied by high-strength bars used on a prestressed wood deck.

Prestressing Holes — predrilled holes at the neutral axis of laminated wood decking used to accommodate prestressing bars on internally-prestressed wood decks.

Prestressing Station/Station — a term referring to the discrete locations at which prestressing is applied (see Internal/External Prestressing Systems).

Sawn Timber — a grading term referring to sawn wood 114 mm or more in least dimension.

Sawn Wood — the product of a sawmill not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing mill, and cross-cutting to length.

Specified Modulus of Elasticity — the modulus of elasticity designated E_{50} is the 50th percentile and E_{05} is the 5th percentile.

Specified Strength — a 5th percentile strength or a calibrated design strength.

Steel/Wood Ratio — the ratio of the cross-sectional area of prestressing steel at each station of a prestressed wood deck to the product of the spacing between stations and the depth of the wood deck.

Stressing Procedure — the procedure outlining the stressing intervals for a prestressed wood deck.

Stretcher — a horizontal member of a crib whose longitudinal axis runs parallel to the long side of the crib.

Transversely-Laminated Deck — a laminated wood deck supported by stringers or beams where the laminations are nominally oriented perpendicular to the direction of the span of the bridge.

Visually Stress-Graded Lumber — structural lumber that has been graded in accordance with the NLGA Standard Grading Rules for Canadian Lumber.

Wood-Concrete Composite Deck — a longitudinally-laminated wood deck composite with a reinforced concrete overlay.

Wood Mudsill — a horizontal member bearing on soil, used to distribute vertical loads.

Wood Trestle — a bridge containing abutments and a series of pile bents and/or framed bents.

Wood Preservative — a chemical formulation meeting the requirements of CSA Standard 080 as being toxic to fungi, insects, borers, and other wood-destroying organisms as restricted by this section.

Notation 13—3

A	cross-sectional area of member or bearing area, mm^2
A_s	total cross-sectional area of prestressing steel at one station, mm^2
A_r	ratio A_s/A
A_w	product of the distance between two consecutive prestressing stations and the depth of wood deck, mm^2
A_b	area of direct bearing on the edge lamination, mm^2
a	length of split at end of member, measured from centreline of support to tip of split, mm
b	width of member or lamination, mm
b_b	width of the distribution of the prestress forces at the edge lamination, mm
b_d	width of prestressed wood deck required to resist the factored load effects, mm
C_k	intermediate slenderness factor
C_m	a factor relating the actual moment diagram to an equivalent uniform moment diagram
C_s	slenderness factor
D_b	diameter of butt of pile, mm
D_h	diameter of prestressing hole, mm
D_t	diameter of tip of pile, mm
d	depth of member, mm
E	modulus of elasticity, MPa
E_{05}	the 5th percentile of modulus of elasticity, MPa
E_{50}	the 50th percentile of modulus of elasticity, MPa
e_o	unamplified eccentricity at the critical section, mm
e_b	unamplified eccentricity at the middle of the unsupported length due to bow in a column, mm

f_{bu}	specified modulus of rupture, MPa
f_{pu}	specified unit compressive strength parallel to grain, or specified tensile strength of prestressing tendon, MPa
f_{py}	specified yield strength of prestressing tendon as defined in CSA Standard G279
$f_{q\ell}$	limiting pressure perpendicular to grain, MPa
f_{qu}	specified unit compressive strength perpendicular to grain, MPa
f_{tu}	specified unit tensile strength parallel to grain, MPa
f_{vu}	specified unit shear strength, for a unit volume of 1 m^3 under uniform shear, MPa
I	moment of inertia of section, mm^4
I_b	moment of inertia of pile at butt, mm^4
k	effective length factor
k_b	modification factor for bearing
k_d	modification factor for duration of load
$k\ell$	modification factor for lateral stability
k_m	modification factor for load-sharing
k_s	modification factor for size effect
k_u	ratio of effective length to total length of pile
k_v	modification factor for shear
L	actual length of component, mm
L_b	length of the distribution of the prestress force along the edge lamination, mm
L_e	effective length of component, mm
L_p	length of steel anchorage plate, mm
L_u	laterally-unsupported length of component, mm
M_o	the factored maximum unamplified moment for columns other than tapered piles, N·mm
M_1	the factored minimum end moment; to be taken as positive when M_1 and M_2 produce single curvature, and negative when they produce double curvature, N·mm
M_2	the factored maximum end moment; always taken as positive, N·mm
M_c	the factored amplified moment resulting from the factored axial load acting in conjunction with M_o, N·mm
M_p	the factored unamplified moment at the critical section in a pile, N·mm
M_u	the factored resistance of a member in flexure, N·mm
M_x	the amplified moment about the X axis for compression members, N·mm
M_y	the amplified moment about the Y axis for compression members, N·mm
M_{xu}	the factored resistance in bending about the X axis of a compression member, N·mm
M_{yu}	the factored resistance in bending about the Y axis of a compression member, N·mm
N_f	assumed uniformly-distributed normal pressure after losses, MPa
N_i	assumed uniformly-distributed normal pressure at transfer, MPa
P	the factored axial load, N
P_u	the factored resistance in compression of an axially loaded short column, N
P_{cr}	the factored Euler buckling capacity, N
R_u	the factored resistance in bearing, N
r	the radius of gyration, mm
S	the elastic section modulus, mm^3
S_b	section modulus of a transformed section of composite wood-concrete deck with respect to the bottom fibres, mm^3
S_t	section modulus of a transformed section of composite wood-concrete deck with respect to the reinforcing steel, mm^3
s	spacing of prestressing stations, mm
T_u	the factored resistance in tension, N
t_p	thickness of anchorage plate, mm
V	the volume of a beam, m^3
V_u	the factored resistance in shear, N
V_{tu}	the factored resistance in vertical shear in the transverse direction of a prestressed wood deck per metre length of the deck, kN
w	width of steel anchorage plate, mm
δ	the moment amplification factor, defined in Clause 13—8.4.1.
η	a factor used in computing P_{cr} for tapered piles
θ	the angle between the plane of loading and the direction of grain
ϕ	a performance factor
μ	coefficient of friction

Limit States 13—4

General **13—4.1**

Structural components shall be proportioned to satisfy the requirements of the serviceability limit state Type I in accordance with Clause 13—4.2 and the ultimate limit state in accordance with Clause 13—4.3.

Serviceability Limit State of Deformation **13—4.2**

The deflection of a flexural component shall not exceed 1/400 of the span under the serviceability limit state Type I load, dynamic load allowance and dead load being excluded. E_{50} in accordance with Tables 13—11.2(a), 13—11.2(b), and 13—12.2 shall be used in calculating the deflection.

Ultimate Limit State of Strength **13—4.3**

Structural components shall be proportioned to have factored resistance greater than or equal to the sum of factored load effects in accordance with the provisions of Section 2.

Performance Factor **13—4.4**

13—4.4.1

The performance factor ϕ for wood shall be as shown in Table 13—4.4.

Table **Performance Factor ϕ** **13—4.4**

	Load Effect				
Component Type	Flexure	Longitudinal Shear	Compression Parallel to Grain	Compression Perpendicular to Grain	Tension Parallel to Grain
Dimension Lumber	0.9	0.8	0.9	1.0	0.9
Sawn Timber	0.9	0.8	0.9	1.0	0.6
Glued-Laminated Timber	0.8	0.9	0.9	1.0	0.7
Piles	0.9	N/A	0.9	N/A	N/A

13—4.4.2

Connections

The performance factor for connections shall be 0.8 in permanent structures, but may be increased to 1.0 for structures with an intended service life not exceeding two years.

General Design 13—5

Design Assumption **13—5.1**

Only elastic analysis, in accordance with the provisions of Section 3 shall be used to determine the force distribution and displacements in the structure.

Spans **13—5.2**

The span length of simply-supported components shall be the distance face-to-face of supports plus one-half the required length of bearing at each end. For continuous members, the span shall be the distance between centres of supports.

Duration of Load Effect **13—5.3**

The modification factor k_d shall be taken as 1.0 except for the following load conditions: dead load alone, earth pressure alone, dead load and earth pressure, for each of which k_d shall be taken as 0.7.

Size Effect **13—5.4**

The specified strength shall be modified for size effect in accordance with Clauses 13—6.2.1 and 13—6.2.2 for flexure, 13—7.1.2 and 13—7.2.2 for shear, 13—9.2 for tension, and 13—10.3 for compression perpendicular to grain.

Moisture Content Effect **13—5.5**

The specified strength and modulus of elasticity values given in this section shall be used for both wet and dry conditions.

Load-Sharing Effect **13—5.6**

The load-sharing factor k_m shall be in accordance with Table 13—5.6.

Notched Components **13—5.7**

Notches or abrupt changes in section shall not be used unless a detailed assessment of the stress concentration effect has been made.

Flexure 13—6

Resistance **13—6.1**

The factored resistance of a flexural component shall be:

$$M_u = \phi f_{bu} S k_d k_\ell k_m k_s$$

where f_{bu} shall be in accordance with Tables 13—11.2(a), 13—11.2(b), 13—12.2, and 13—13.3.

Table **Load-Sharing Factor k_m** **13—5.6**

Components	Grade	
	No. 1 & 2	S.S.
Nailed, vertically-laminated decks	1.20	1.20
Prestressed, vertically-laminated decks	1.50	1.30
Wood-concrete composite decks	1.40	1.20
Other components spaced not more than 600 mm apart	1.10	1.10

Size Effect **13—6.2**

13—6.2.1

Sawn Wood

The size effect factor k_s for dimension lumber and for beams and stringers in flexure shall be in accordance with Table 13—6.2.1.

Table **Size Effect Factor k_s for Flexure for all Grades and Species** **13—6.2.1**

Depth mm	89	140	184	235	286	337	389 and larger
Factor	1.5	1.3	1.2	1.1	1.0	0.9	0.8

13—6.2

Glued-Laminated Beams

The size effect factor k_s for glued-laminated beams in flexure shall be

$$k_s = V^{-0.086}$$

where V is the beam volume in the span in m³.

Duration of Load and Load-Sharing Effects **13—6.3**

The factors k_d for duration of load and k_m for load-sharing shall be in accordance with Clauses 13—5.3 and 13—5.6, respectively.

Lateral Stability **13—6.4**

13—6.4.1 When the depth of a beam does not exceed its breadth, no lateral support is required and k_ℓ shall be taken as 1.0.

13—6.4.2 When the depth of a beam exceeds its breadth, lateral support shall be provided at the points of bearing so as to prevent torsional rotation.

13—6.4.3 When the depth of a beam exceeds its breadth, the factored resistance in flexure shall be modified in accordance with Clauses 13—6.5 to 13—6.8 inclusive. When the compression edge is effectively supported along its length, no modification is required.

Slenderness Factor of a Beam **13—6.5**

The slenderness factor of a beam shall be calculated as follows:

$$C_s = \sqrt{\frac{L_u d}{b^2}}$$

Short Beams **13—6.6**

When the slenderness factor of a beam does not exceed 6, k_ℓ shall be taken as 1.0.

Intermediate Beams **13—6.7**

When the slenderness factor is greater than 6, but does not exceed C_k, the slenderness modification factor for lateral stability k_ℓ shall be determined as follows:

$$k_\ell = 1 - 0.3\left[\frac{C_s}{C_k}\right]^4$$

where $$C_k = \sqrt{\frac{E_{05}}{f_{bu}}}$$

Slender Beams **13—6.8**

When the slenderness factor exceeds C_k, the modification factor k_ℓ, for lateral stability shall be determined as follows:

$$k_\ell = \frac{0.70\,E_{05}}{C_s^2\,f_{bu}}$$

In no case shall a beam have a slenderness Factor, C_s, exceeding 30.

Shear **13—7**

The critical section for longitudinal shear in rectangular wood beams shall be taken at a distance from the support equal to the depth of the beam. Longitudinal shear shall be neglected in vertically-laminated decks.

Sawn Wood **13—7.1**

13—7.1.1 **Resistance**

The factored resistance in shear of a sawn rectangular beam shall be taken as:

$$V_u = \frac{\phi f_{vu} bd}{1.5} k_d \, k_m \, k_v$$

where f_{vu} shall be in accordance with Table 13—11.2(a) and 13—11.2(b), and k_d, k_m and k_v shall be in accordance with Clauses 13—5.3, 13—5.6, and 13—7.1.2, respectively.

13—7.1.2 **Size Effects**

13—7.1.2.1 **New Structures**

The size effect factor k_v for shear at the end of a sawn beam in new structures shall be:

$k_v = \frac{45}{\sqrt{d}}$ for wood graded as select structural and for No. 1 when graded as select structural for end splits.

$k_v = \frac{30}{\sqrt{d}}$ for wood graded as No. 1.

13—7.1.2.2 **Existing Structures**

Where the length of the end split is known, k_v shall be:

$$k_v = \frac{75}{\sqrt{d}} \cdot \frac{1}{1 + 2a/d}$$

except that for bridges less than 5 years old k_v shall not exceed that given for new structures.

Glued-Laminated Timber **13—7.2**

13—7.2.1 **Resistance**

The factored resistance of a glued-laminated prismatic rectangular beam in shear shall be:

$$V_u = \frac{\phi f_{vu} bd}{1.5} k_d \, k_m \, k_v$$

where f_{vu} values shall be in accordance with Table 13—12.2 and k_v shall be in accordance with Clause 13—7.2.2.

13—7.2.2 **Size Effects**

13—7.2.2.1 **New Structures**

The modification factor k_v for shear of glued-laminated beams in new structures shall be taken as:

$$k_v = 2.3V^{-0.15}$$

where V is the beam volume in the span in m^3.

13—7.2.2.2 **Existing Structures**

For analysis of existing structures in which the length of the end split is known, k_v shall be the lesser of the value obtained in accordance with Clause 13—7.2.2.1 and

$$k_v = \frac{75}{\sqrt{d}} \cdot \frac{1}{1 + 2a/d}$$

Duration of Load and Load-Sharing Effects **13—7.3**

The factors k_d for duration of load and k_m for load-sharing shall be in accordance with Clauses 13—5.3 and 13—5.6, respectively.

Compression members **13—8**

All compression members shall be designed for combined compression and bending at ultimate limit states.

Resistance **13—8.1**

The factored resistance in compression for use in the interaction formulae shall be taken as:

$$P_u = \phi A f_{pu} \, k_d \, k_m$$

where f_{pu} shall be in accordance with Tables 13—11.2(a), 13—11.2(b), and 13—12.2, and k_d and k_m shall be in accordance with Clauses 13—5.3 and 13—5.6, respectively.

Slenderness Effect in Compression Members **13—8.2**

13—8.2.1

The design of compression members shall take into account the effect of factored axial loads in amplifying the moments due to end eccentricities, bow, and lateral loads in the unsupported length L_u. The end eccentricities and bow shall be in accordance with Clauses 13—8.5.1, 13—8.6, and 13—8.8.3.

13—8.2.2 In lieu of Clause 13—8.2.1, the design of compression members may be based on an approximate procedure in accordance with Clause 13—8.4.

Evaluation of Slenderness Effect

13—8.3

13—8.3.1 **Unsupported Length**

The unsupported length L_u of a compression member shall be taken as the distance, centre-to-centre, of lateral supports each capable of providing a lateral restraint force of at least 4% of the axial load on the member. Such lateral restraint force shall also be not less than that required to resist the effects of end moments and lateral loading.

13—8.3.2 **Effective Length**

Compression members shall be designed on the basis of their effective length $L_e = kL_u$.

13—8.3.3 **Effective Length of Members Braced Against Side-Sway**

For compression members braced against side-sway, the effective length factor k shall be taken as 1.0 unless an analysis shows that a lower value may be used.

13—8.3.4 **Effective Length of Members Not Braced Against Side-Sway**

For compression members not braced against side-sway, the effective length factor k shall not be less than the value shown in Table A13—1 where applicable, or as determined by an analysis, but shall not be less than 1.0.

13—8.3.5 **Conditional Limitations**

a) For compression members not braced against side-sway, the effects of slenderness may be neglected when:

$$\frac{kL_u}{r} \text{ is less than } 40$$

b) For compression members braced against side-sway, the effects of slenderness may be neglected when:

$$\frac{kL_u}{r} \text{ is less than } 60 - 20\frac{M_1}{M_2}.$$

c) $\frac{kL_u}{r}$ shall not exceed 120.

13—8.3.6 **Interaction for Uniaxial Bending**

Compression members with uniaxial bending shall satisfy:

$$\frac{P}{P_u} + \frac{M_c}{M_u} \leq 1.0$$

Approximate Method for Compression Members

13—8.4

13—8.4.1 In conjunction with Clause 13—8.3 the following approximate method of determining M_c may be used:

$$M_c = \delta M_0 \text{ but not less than } M_2$$

where:

$$\delta = \frac{C_m}{1 - P/P_{cr}}$$

$$P_{cr} = \phi \frac{\pi^2 E_{05} I}{(kL_u)^2} k_d k_m$$

13—8.4.2 For members braced against side-sway and without transverse loads between supports, C_m in Clause 13—8.4.1 may be taken as:

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \text{ but not less than } 0.4.$$

13—8.4.3 For all other cases C_m shall be taken as 1.0.

End Moments

13—8.5

13—8.5.1 All compression members shall be analysed for end eccentricities at each end equal to the greater of:

a) those corresponding to the maximum end moments associated with the axial load; or

b) 0.05 times the lateral dimension of the compression member in the plane of flexure being considered.

The end eccentricity in (b) shall be taken to cause uniaxial bending and single curvature only.

Bow Moments **13—8.6**

All compression members, except piles, shall be analysed for the load effect due to an eccentricity:

$$e_b = \frac{L_u}{500}$$

midway between points of lateral support. Bow moments shall be assumed to act in the same plane and with the same sense as the end moment derived from Clause 13—8.5.1.

Biaxial Bending **13—8.7**

13—8.7.1 **Moment Magnifiers**

When compression members are subject to bending about both principal axes the moment about each axis shall be amplified by δ, computed from the corresponding conditions of restraint and the flexural stiffness about the axis.

13—8.7.2 **Interaction for Biaxial Bending**

Compression members with biaxial bending shall satisfy

$$\frac{P}{P_u} + \frac{M_x}{M_{xu}} + \frac{M_y}{M_{yu}} \leq 1.0$$

Slenderness Effect for Piles **13—8.8**

13—8.8.1

Piles shall be designed in accordance with Clause 13—8.3.6. The following approximate method of determining M_c may be used.

$$M_c = \delta M_p \text{ but not less than } M_2$$

where: M_p is determined in accordance with Clause 13—8.8.3

$$\delta = \frac{C_m}{1 - P/P_{cr}}$$

$$P_{cr} = \phi \frac{\eta E_{05} I_b}{(kL_u)^2} k_d k_m$$

where η is in accordance with Table A13—3.

13—8.8.2 **Effective Length of Piles**

When the finished pile projects above the ground level and is not secured against buckling by adequate bracing, the effective length shall be governed by the restraint conditions imposed on it by the structure it supports and by the nature of the ground into which it is driven. In firm ground, the lower point of contraflexure may be taken at a depth below ground level of one-tenth of the exposed length. Where the top stratum is soft clay or silt, the point of contraflexure may be taken at one-half the depth of penetration into this stratum but not less than one-tenth of the exposed length of pile. Where a pile is wholly embedded in soil, the effect of slenderness shall be ignored. A stratum of extremely soft soil, peat, or mud shall be treated as if it were water.

13—8.8.3

When lateral displacement of the pile butt is prevented, M_p shall be determined at a section 0.55 times the effective length below the butt, as the product of P and e_0, plus the effects of any end moments and lateral loads. Values of e_0 shall be in accordance with Table A13—2.

Duration of Load and Load Sharing Effects **13—8.9**

The factors k_d for duration of load and k_m for load sharing shall be in accordance with Clauses 13—5.3 and 13—5.6, respectively.

Tension **13—9**

Resistance **13—9.1**

The factored resistance of a tension member shall be determined for both net and gross sections as follows:

$$T_u = \phi f_{tu} A k_d k_s k_m$$

Size Effect **13—9.2**

The modification factor k_s for dimension lumber shall be in accordance with Table 13—9.2 and shall only apply when determining the factored resistance of the net section.

Table **Size Effect Factor for Tension k_s** **13—9.2**

Depth mm	89	114	140	184	235	286 and larger
All Species All Grades	1.25	1.20	1.15	1.10	1.05	1.0

Duration of Load and Load-Sharing Effects **13 — 9.3**

The factors k_d for duration of load and k_m for load sharing shall be in accordance with Clauses 13 — 5.3 and 13 — 5.6, respectively.

Compression or Bearing Not Parallel to Grain 13 — 10

General **13 — 10.1**

The factored resistance in bearing perpendicular to grain shall be:

$$R_u = \phi A f_{qu} k_b k_d k_m$$

where f_{pu} shall be in accordance with Tables 13 — 11.2(a), 13 — 11.2(b), and 13 — 12.2.

Loading at an Angle to the Grain **13 — 10.2**

The factored resistance in bearing normal to a plane inclined to the grain shall not exceed that determined by:

$$R_u = \phi \frac{A f_{pu} f_{qu}}{f_{pu} \sin^2\theta + f_{qu} \cos^2\theta} k_b k_d k_m$$

where ϕ shall be taken as 0.9 and f_{pu} and f_{qu} shall be in accordance with Tables 13 — 11.2(a), 13 — 11.2(b), and 13 — 12.2.

Size Effect **13 — 10.3**

When lengths of bearing or diameters of washers are less than 150 mm and no part of the bearing area is closer to the end of the member than 75 mm, the factored resistance in bearing may be modified by the factor k_b shown in Table 13 — 10.3 provided that such bearing areas do not occur in positions of high flexural stress.

Table **Modification Factors for Specified Strength in Bearing, k_b** **13 — 10.3**

Length of Bearing, mm	15	25	40	50	75	100	150 or more
Factor	1.75	1.38	1.25	1.19	1.13	1.10	1.00

Duration of Load and Load-Sharing Effects **13 — 10.4**

The factors k_d for duration of load and k_m for load sharing shall be in accordance with Clauses 13 — 5.3 and 13 — 5.6, respectively.

Table **Species Combinations** **13 — 11.1.1**

Individual Species or Species Combination	Stamp Identification	Species Included in the Combination
Douglas Fir-Larch	D Fir-L(N)	Pacific Coast Douglas Fir*, Mountain Douglas Fir, Western Larch*
Hem-Fir	Hem-Fir(N)	Western Hemlock**, Amabilis Fir*, Grand Fir
Spruce-Pine-Fir	S-P-F or Spruce-Pine-Fir	Spruce (all species except Coast Sitka Spruce), Jack Pine*, Lodgepole Pine*, Balsam Fir, Alpine Fir, Ponderosa Pine*
Jack Pine	J Pine	Jack Pine*
Lodgepole Pine or Ponderosa Pine	L Pine or P Pine	Lodgepole Pine* Ponderosa Pine*
Red Pine	Red Pine	Red Pine*
White Pine	WW Pine or EW Pine	Western White Pine* Eastern White Pine*

Notes: Only the species marked with an asterisk (*) are suitable for pressure preservative treatment. Species marked (**) are treatable with some difficulty.

Sawn Wood 13—11

Materials 13—11.1

13—11.1.1 Species and Species Combinations

Only individual species or species combinations in accordance with Table 13—11.1.1 shall be used.

13—11.1.2 Grades of Sawn Wood

All soft wood shall be visually stress-graded in conformity with the NLGA Standard Grading Rules for Canadian Lumber, and shall comply with CSA Standard 0141, *Softwood Lumber*.

13—11.1.3 Identification of Wood

All wood shall be identified by a grade stamp of an association or independent grading agency approved by the Canadian Lumber Standards Administrative Board in accordance with the provisions of CSA Standard 0141. Where preservative treatment may obscure the grade mark, a certificate of inspection or other approved evidence of grade shall be supplied by the treating company.

Table Specified Strength and Modulus of Elasticity for Dimension Lumber, Thickness 38 mm to 77 mm, MPa** 13—11.2(a)

Species or Species Combination	Grade	Bending f_{bu}	Shear Longitudinal f_{vu}	Compression		Tension	Modulus of Elasticity	
				Parallel to Grain f_{pu}	Perpendicular to Grain f_{qu}	Parallel to Grain f_{tu}	E_{50}	E_{05}
Douglas Fir Larch	Select Structural	17.5	1.1	16.5	3.6	13.5	11 000	8 000
	No. 1 and No. 2	10.0	1.1	9.0	3.6	9.0	9 500	6 000
Hem-Fir	Select Structural	16.0	0.8	14.5	1.9	13.5	11 000	7 500
	No. 1 and No. 2	11.5	0.8	10.5	1.9	9.0	10 500	7 000
S.P.F.	Select Structural	16.0	0.8	14.5	1.9	13.5	10 000	7 000
	No. 1 and No. 2	11.5	0.8	10.5	1.9	9.0	9 000	6 000
Lodgepole Pine Pondersosa Pine	Select Structural	16.0	1.0	14.5	1.9	13.5	10 000	7 000
	No. 1 and No. 2	11.5	1.0	10.5	1.9	9.0	9 000	6 000
Jack Pine	Select Structural	16.0	1.0	14.5	2.6	13.5	10 500	7 000
	No. 1 and No. 2	11.5	1.0	10.5	2.6	9.0	9 500	6 000
Red Pine	Select Structural	11.5	0.8	10.0	1.9	10.0	7 000	5 000
	No. 1 and No. 2	8.0	0.8	7.0	1.9	7.0	6 000	4 000
White Pine*	Select Structural	6.0	0.8	—	1.9	—	5 500	4 000
	No. 1 and No. 2	4.5	0.8	—	1.9	—	4 500	3 500

---

* For use in prestressed vertically-laminated decks only.

** Dimension lumber with thickness of 89 mm or greater shall have specified strengths in accordance with Table 13—11.2(b).

Footnote: The value of shear modulus may be estimated as 0.065 times the modulus of elasticity E_{50}.

Table **Specified Strength and Modulus of Elasticity for Beams & Stringers, Posts & Timbers, Minimum Dimension 140 mm, MPa** **13 — 11.2(b)**

Species or Species Combination	Grade	Bending f_{bu}	Shear Longitudinal f_{vu}	Compression		Tension	Modulus of Elasticity	
				Parallel to Grain f_{pu}	Perpendicular to Grain f_{qu}	Parallel to Grain f_{tu}	E_{50}	E_{05}
Douglas Fir Larch	Select Structural	24.0	1.1	16.5	3.6	13.0	11 000	7 500
	No. 1	20.0	1.1	9.0	3.6	9.0	9 500	6 500
Hem-Fir	Select Structural	20.0	0.8	14.5	1.9	13.0	11 000	7 500
	No. 1	18.0	0.8	10.5	1.9	9.0	10 500	7 000
S.P.F.	Select Structural	18.5	0.8	14.5	1.9	13.0	10 000	7 000
	No. 1	13.0	0.8	10.5	1.9	9.0	9 000	6 000
Lodgepole Pine Ponderosa Pine	Select Structural	18.5	1.0	14.5	1.9	13.0	10 000	7 000
	No. 1	13.0	1.0	10.5	1.9	9.0	9 000	6 000
Jack Pine	Select Structural	18.5	1.0	14.5	2.6	13.0	10 500	7 000
	No. 1	13.0	1.0	10.5	2.6	9.0	9 500	6 000
Red Pine	Select Structural	13.0	0.8	10.0	1.9	10.0	7 000	5 000
	No. 1	9.0	0.8	7.0	1.9	7.0	6 000	4 000

Footnote: The value of shear modulus may be estimated as 0.065 times the modulus of elasticity E_{50}.

Specified Strength and Modulus of Elasticity **13 — 11.2**

Specified strength and moduli of elasticity for sawn wood shall in accordance with Tables 13 — 11.2(a) and 13 — 11.2(b).

Structural Glued-Laminated Timber 13 — 12

Materials **13 — 12.1**

13 — 12.1.1

All structural glued-laminated timber shall be exterior service grade and manufactured in accordance with CSA Standard 0122, *Structural Glued-Laminated Timber*, by a plant certified under CSA Standard 0177, *Qualification Code for Manufacture of Structural Glued-Laminated Timber*.

24f-E — Bending Grade shall be specified for simple span beams and for compression members subjected to substantial bending moment.
24f-EX — Bending Grade shall be specified for continuous beams.
16c-E — Compression Grade shall be specified for members primarily stressed in axial compression.
18t-E — Tension Grade shall be specified for members primarily stressed in axial tension.

Specified Strength and Modulus of Elasticity **13 — 12.2**

Specified strength and modulus of elasticity for structural glued-laminated timber shall be in accordance with Table 13 — 12.2.

Table **Specified Strength and Modulus of Elasticity for Glued-Laminated Douglas Fir, MPa** **13 — 12.2**

Type of Stress		CSA Stress Grade			
		24f-E Bending Grade	24f-EX Bending Grade	16c-E Compression Grade	18t-E Tension Grade
Bending at extreme fibre due to positive bending movement	f_{bu}	23.4	23.4	11.1	18.4
Bending at extreme fibre due to negative bending movement	f_{bu}	11.1	23.4	11.1	18.4
Longitudinal shear	f_{vu}	1.1	1.1	1.1	1.1
Compression parallel to grain	f_{pu}	16.5	16.5	16.5	19.3
Compression perpendicular to grain	f_{qu}	3.6	3.6	3.6	3.6
Axial tension at gross section	f_{tu}	10.5	10.5	10.5	12.8
Axial tension at net section	f_{tu}	14.1	14.1	14.1	16.5
Modulus of elasticity	E_{50}	11 700	11 700	11 000	12 400
	E_{05}	10 000	10 000	9 400	10 500

Vertically-Laminated Beams **13 — 12.3**
The factored resistance in flexure for beams composed of vertical laminations shall be calculated as for load-sharing systems in sawn wood.

Camber **13 — 12.4**
Glued-laminated beams shall be cambered for the sum of 1/600 of the span and twice the calculated deflection due to the unfactored dead loads.

Varying Depth **13 — 12.5**
Variation in the depth of a flexural member is permitted provided that the bevel of the laminations on the tension side is not steeper than 7% and that the fibre stress is less than 50% of the specified strength.

Curved Members **13 — 12.6**
The provisions of this section shall apply only to glued-laminated members with a radius greater than 12 m and the reduction in capacity due to curvature may be ignored.

Wood Piling **13 — 13**

Materials **13 — 13.1**
Wood piling, including Southern Yellow Pine, shall conform to the requirements of CSA Standard CAN3-056M, *Round Wood Piles*. Mountain Douglas Fir, Pacific Coast Hemlock, Spruce, and Western Red Cedar shall not be used where preservative treatment is required.

Splicing **13 — 13.2**
Wood piles shall not be spliced unless the splice detail is approved.

Specified Strength and Modulus of Elasticity **13 — 13.3**
Specified strength and modulus of elasticity for round wood piles shall be in accordance with Table 13 — 13.3.

Design **13 — 13.4**

13 — 13.4.1 **General**
Wood piling shall be in accordance with the provisions of Section 6.

13 — 13.4.2 **Embedded Portion**
Subject to the ability of the pile to sustain the loads, the portion of pile permanently in contact with a soil mass which provides adequate lateral support shall be designed

as a short column using the specified strength in accordance with Table 13—13.3. The factored resistance of an end-bearing pile shall be calculated on the basis of the minimum cross-section. The factored resistance of a friction pile shall be calculated on the basis of the cross-section located one-third of the thickness of the supporting stratum above the tip.

13—13.4.3 **Unembedded Portion**

The portion of the pile in contact with air, water, or a soil mass not providing adequate lateral support, shall be designed as a tapered column in accordance with Clause 13—8.8 using the specified strength given in Table 13—13.3.

Table **Specified Strength and Modulus of Elasticity for Round Wood Piles, MPa** **13—13.3**

Species	Stress in Bending f_{bu}	Compression Parallel to Grain f_{pu}	Modulus of Elasticity E_{05}
Douglas Fir	26	26	9 500
Western Larch	26	26	9 500
Pacific Coast Hemlock	22	22	9 300
Jack Pine	19	19	5 500
Lodgepole Pine	19	19	5 500
Ponderosa Pine			
Red Pine	19	19	6 500
Spruce	16	16	5 000
Southern Yellow Pine	19	19	7 500
Western Red Cedar	18	18	5 000

Fastenings 13—14

Materials 13—14.1

13—14.1.1 **Fastenings for Wood**

Fastenings for wood and their installation shall conform to CSA Standard CAN3-086M, *Code for Engineering Design in Wood*.

13—14.1.2 **Temporary Structures**

All fastenings specified in CSA Standard CAN3-086M shall be permitted, where the intended service life of a structure does not exceed two years.

13—14.1.3 **Permanent Structures**

Glulam rivets and truss plates shall not be used in permanent structures, except as specified for wood-concrete composite decks in Clause 13—21.2.1.3.

Specified Strength **13—14.2**

The nominal resistance for fastenings shall be taken as 1.7 times the allowable loads as determined by the requirements of Clause 9 of CSA Standard CAN3-086M. In the design of fastenings, the wood shall be assumed to be seasoned at the time of fabrication and to be used under wet service conditions.

Factored Resistance **13—14.3**

The applicable modification factor for duration of load in accordance with Clause 13—5.3, and the appropriate performance factor in accordance with Clause 13—4.4.2, shall be applied.

Hardware and Metal Work 13—15

General **13—15.1**

All steel plates, shapes, and welded assemblies shall be in accordance with the provisions of Section 10.

Protective Treatment **13—15.2**

All hardware and metal work used in permanent structures, including fastenings (for example, drift pins, split rings, shear plates, nails, prestressing materials), shall be hot-dipped galvanized in accordance with CSA Standard G164, *Hot-Dip Galvanizing of Irregularly-Shaped Articles*. Galvanized nuts shall be retapped to allow for the increased diameter of the bolt due to galvanizing. Heat-treated alloy components and fastenings shall be protected by an approved alternative protective treatment.

Pressure Preservative Treatment 13—16

General **13—16.1**

Except as provided for in Clauses 13—16.2, and 13—16.3, all wood in permanent structures shall be oil-borne pressure preservative treated in accordance with CSA Standard 080, *Wood Preservation*, and only the following preservatives shall be used:

- creosote;
- pentachlorophenol in Type A hydrocarbon solvent.

Net retentions of preservatives shall be the minima as specified in CSA Standard 080 for the appropriate conditions.

The use of other preservative treatments for main structural members is subject to approval.

Pedestrian Contact **13 — 16.2**

Main structural members shall not be exposed to direct pedestrian contact. For components other than main structural members (such as handrails, posts and curbs) subject to direct pedestrian contact, the preservative used shall be Chromated Copper Arsenate (CCA) or Ammoniacal Copper Arsenate (ACA). Net retentions of preservatives shall be the minima as specified in CSA Standard 080 for the appropriate conditions.

Direct pedestrian contact is considered to be contact which can be made while the pedestrian is situated anywhere in the access route provided for pedestrain traffic. Where a sidewalk is provided which is separated by a barrier from the roadway, the pedestrian access route may be taken as the sidewalk area only. Where no sidewalk is provided, or the sidewalk is not separated from the roadway, the pedestrian access route shall be taken as the area bounded by the guiderails.

Untreated Piles **13 — 16.3**

Untreated wood foundation piles may be used in permanent structures provided that they are clean-peeled and free from wood-destroying organisms, that the cut-off is below a known permanent water level, and that the pile is completely embedded in soil.

Incising **13 — 16.4**

Sawn wood and glued-laminated members shall be incised prior to preservative treatment, in accordance with CSA Standard 080.

Glued-laminated members too large to be mechanically incised shall be incised by hand, throughout the area of contact with caps or sills and hold-down brackets. These requirements shall be noted on the plans.

Fabrication **13 — 16.5**

Except for fabrication that cannot be accurately detailed prior to erection, all treated wood shall be cut to finished size; all surfacing, holes, notches, ring grooves, chamfering, daps, and other cuts shall be made prior to pressure preservative treatment. All other cuts made after preservative treatment shall have the resultant exposed surfaces field treated in accordance with the standard for the Care of Preservative Treated Wood Products, as specified in CSA Standard 080. These requirements shall be noted on the plans.

Wood Cribs **13 — 17**

General **13 — 17.1**

13 — 17.1.1 Wood cribs shall be assumed to act as a unit and shall be designed to resist overturning and sliding in accordance with the provisions of Section 6.

13 — 17.1.2 Headers and stretchers shall be designed to resist the bending and shearing load effects imposed on them and to provide adequate bearing.

13 — 17.1.3 Vertical spacing between members shall be small enough to retain the fill. The crib shall be closed-faced where an ice problem is anticipated.

Member Sizes and Assembly **13 — 17.2**

13 — 17.2.1 Members for wood cribs shall have a minimum dimension of 184 mm and stretchers shall be as long as practicable to achieve continuity.

13 — 17.2.2 Joints in each tier of the crib shall be staggered with respect to joints in adjacent tiers.

Fastening **13 — 17.3**

Members shall be connected by means of drift pins not less than 19 mm in diameter and of sufficient length to extend completely through one tier and at least three quarters through the next tier.

Bearings **13 — 17.4**

Bearings shall be provided to minimize the effects of eccentric loads from the superstructure.

Wood Trestles **13 — 18**

General **13 — 18.1**

Piles for trestles shall be designed in accordance with Clauses 13 — 8 and 13 — 13. Tops of piles, not otherwise encased, shall be fitted with snug steel collars having minimum dimensions of 5 × 75 mm.

Caps, sills, and decks shall be securely fastened to resist uplift forces due to bouyancy, in accordance with the provisions of Section 2.

Pile Bents **13 — 18.2**

Pile bents higher than 3.0 m shall be braced transversely in accordance with Clause 13 — 18.5. Longitudinal bracing shall be provided unless a detailed analysis shows that it may be omitted.

Framed Bents **13 — 18.3**

13 — 18.3.1 Framed bents shall be supported on piles, concrete pedestals, or mudsills. All bents shall be braced transversely and longitudinally in accordance with Clause 13 — 18.5.

13 — 18.3.2 **Sills**

Sills shall be connected to piles or mudsills by drift pins not less than 19 mm in diameter, extending at least 300 mm into the pile and at least 150 mm into the mudsill. Sills shall be connected to concrete pedestals by anchor bolts not less than 19 mm in diameter, spaced at not more than 1.8 m centres.

13 — 18.3.3 **Post Connections**

Posts shall be connected to the sill beam by clip angles, steel dowels or drift pins at least 19 mm in diameter, extending at least 300 mm into the post and at least 150 mm into the sill.

Caps **13 — 18.4** Caps shall be connected to the piles or posts by steel drift pins at least 19 mm in diameter, extending at least 300 mm into the pile or post.

Bracing **13 — 18.5**

13 — 18.5.1 **Transverse Bracing**

Diagonal bracing shall be provided on each side of a bent and shall have a minimum size 76 × 203 mm. The bracing shall be bolted to all posts, piles, caps, and sills. The bolts shall be at least 19 mm in diameter. Where multiple storey bracing is required, horizontal bracing members the same size as the diagonal bracing, shall be placed between tiers.

13 — 18.5.2 **Longitudinal Bracing**

The requirements for longitudinal bracing shall be determined by analysis. The diagonal braces shall have a minimum size of 76 × 203 mm and the longitudinal girts shall have a minimum size of 152 × 203 mm.

Stringers and Girders **13 — 19**

Design Details **13 — 19.1** The plans shall specify that the stringers be sized to permit even bearing and to compensate for variations in stringer depths at supports.

All stringers and girders shall be securely fastened to resist buoyancy effects in accordance with the provisions of Section 2. Bolts or drift pins not less than 19 mm in diameter shall extend at least 150 mm into the cap at each end of the stringer or girder.

Diaphragms **13 — 19.2**

13 — 19.2.1 Stringers and girders shall be provided with diaphragms at each support to transmit lateral forces to the bearings and to prevent rotation about their longitudinal axes.

13 — 19.2.2 Unless otherwise approved, intermediate diaphragms shall be provided at the one-third span points, except that for spans less than 12 m, a single intermediate diaphragm at mid-span shall suffice.

Nailed-Laminated Wood Decks **13 — 20**

General **13 — 20.1** The provisions of this clause are applicable to wood decks composed of nailed-laminated dimension lumber. Where the wood deck surface is exposed to traffic, an allowance of 15 mm to the width of laminates shall be included to account for wear.

Transversely-Laminated Wood Decks **13 — 20.2** The thickness of laminations shall be from 38 mm to 51 mm, and the minimum width shall be 89 mm. The difference in width of the deck laminations shall not exceed 5 mm.

13 — 20.2.1 **Assembly**

13 — 20.2.1.1 **Nailing**

Common nails in accordance with CSA Standard B111-1974, *Wire Nails, Spikes and Staples*, shall be used to fasten each lamination to the preceding one at each end and at approximately 300 mm intervals. The nails shall be driven alternately near the top and bottom edges. The nails shall be of sufficient length to pass through two laminations and at least halfway through the third one. At least one nail shall be placed within 100 mm to 125 mm of the end of each lamination.

13 — 20.2.1.2 **Deck Support Anchorage With Wood Stringers**

Laminations shall be securely fastened to stringers either by bolts, lag screws, lugs, and angles or by each of the laminations being toe-nailed with 100 mm nails as follows:

a) one nail at every support for spacing of stringers or girders not exceeding 1.2 m,

b) two nails at every support for spacing of stringers or girders exceeding 1.2 m.

13 – 20.2.1.3 **Deck Support Anchorage With Steel Stringers**

Laminations shall be connected to the top flanges of steel stringers by galvanized steel nailing clip-plates not less than 2 mm in thickness.

Clip-plates shall be spaced at 450 mm centres on each side of the flange and shall be staggered by 225 mm.

13 – 20.2.1.4 Each lamination shall be vertical, tight against the preceding one, and bearing evenly on all supports.

13 – 20.2.1.5 **Butt Joints**

Butt joints shall be so staggered that within any 1 m wide band (running perpendicular to the direction of the laminations) butt joints shall not occur in more than one lamination in three.

Longitudinally-Laminated Wood Decks **13 – 20.3** Nailed, longitudinally-laminated wood decks shall only be employed when made composite with a concrete overlay in accordance with Clause 13 – 21, or when an approved alternative method of providing load-sharing among the laminations is used.

Wood-Concrete Composite Decks 13 – 21

General **13 – 21.1** The provisions of this clause are applicable to nailed, longitudinally-laminated wood decks made composite with a reinforced concrete overlay.

Wood Base **13 – 21.2** The wood base shall consist of longitudinally-laminated dimension lumber, 38 to 76 mm thick, with widths not less than 140 mm nor more than 292 mm.

13 – 21.2.1 **Assembly**

13 – 21.2.1.1 The provisions of Clauses 13 – 20.2.1.1 and 13 – 20.2.1.4 shall apply.

13 – 21.2.1.2 **Butt Joints**

Butt joints in the wood base shall be staggered in such a manner that within any 1 m wide band (running perpendicular to the direction of the laminations), butt joints shall not occur in more than one lamination in four.

13 – 21.2.1.3 **Spliced Butt Joints**

Butt joints shall be provided with a connection detail as shown in Figure 13 – 21.2.1.3, or another approved equivalent method. Hot-dipped galvanized steel nail plates shall be installed by applying uniform pressure, by hydraulic press or other approved means.

13 – 21.2.1.4 **Deck Anchorage**

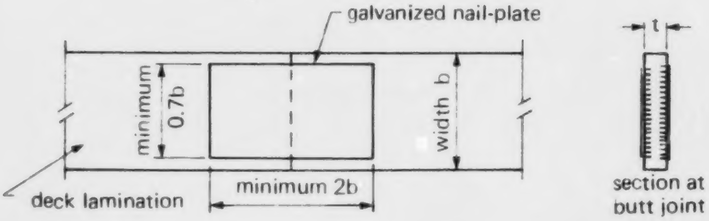
The laminated wood base shall be securely fastened to the supports in order to resist buoyancy in accordance with the provisions of Section 2.

This requirement may be deemed to be satisfied if the laminations are toe-nailed with 100 mm common nails as follows:

a) one nail at each support for each lamination that is continuous over the support and also at abutments.

b) one nail in each butting lamination at joints over the supports.

Figure **Spliced Butt Joint** **13 – 21.2.1.3**



thickness of lamination, t	minimum base steel nominal thickness, mm
38 mm to 45 mm	1.3
46 mm to 51 mm	1.6
52 mm to 76 mm	2.0

Concrete Slab **13 – 21.3**

13 – 21.3.1 **Strength**

The concrete shall have a minimum specified strength of 30 MPa

13 – 21.3.2 **Thickness**

The net thickness of the concrete slab shall not be less than 125 mm

13 – 21.3.3 **Reinforcement**

The minimum reinforcement in the concrete slab shall consist of a mat of size 10 bars placed on 180 mm centres in both directions

In continuous spans, the tensile steel shall be designed to provide the factored flexural resistance Additional reinforcement when necessary shall be placed on top of the mat

Concrete cover to the top of the deck shall conform to the requirements of Section 8

Concrete cover to the wood-concrete interface shall not be restricted, except that in the case of the method specified in Clause 13 – 21 4 2(b), the reinforcement shall not bear directly on the wood base

Wood-Concrete Interface **13 – 21.4**

13 – 21.4.1 **General**

The wood base and the concrete slab shall be connected in such a manner as to prevent separation and to resist the factored horizontal shear forces between the two materials under repeated loads

13 – 21.4.2

The provisions of Clause 13 – 21 4 1 shall be considered satisfied if one of the following methods is employed

a) The wood base consists of laminations which alternate in width by 51 mm to form longitudinal grooves The top surface of all laminations are dapped and the sides of the higher laminations are grooved in accordance with Figure 13 – 21 4 2(a)

b) The wood base consists of laminations of substantially equal width, with variations in width not exceeding 5 mm The top surface of the laminations contain transverse grooves 38 mm deep, 150 mm wide and spaced at approximately 600 mm centre-to-centre Common nails at least 50 mm longer than the width of the laminations shall be driven into alternate laminations to provide shear key reinforcement in accordance with Figure 13 – 21 4 2(b) Where the grooves in adjacent laminations are staggered by more than 50 mm, all laminations involved shall contain shear key reinforcing nails

Figure **13 – 21.4.2(a)**

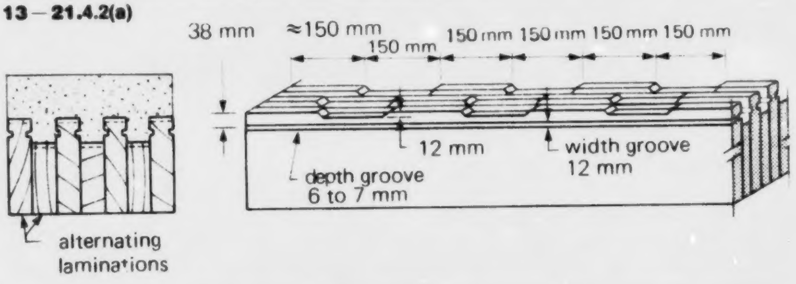
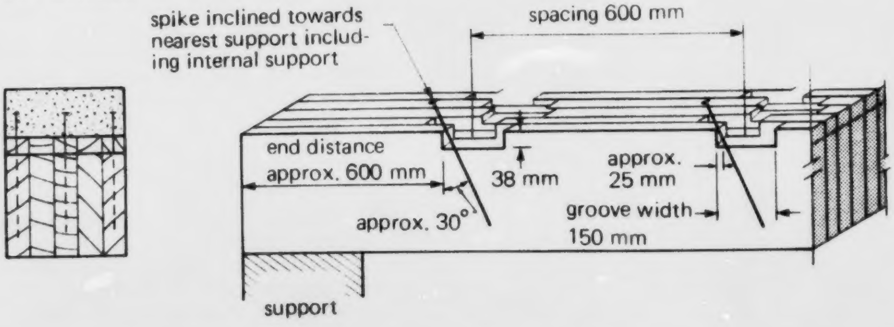

38 mm
≈150 mm
150 mm
150 mm 150 mm 150 mm 150 mm
12 mm
width groove
12 mm
depth groove
6 to 7 mm
alternating
laminations

Figure **13 – 21.4.2(b)**


spike inclined towards
nearest support includ-
ing internal support
spacing 600 mm
end distance
approx. 600 mm
38 mm
approx.
25 mm
approx. 30°
groove width
150 mm
support

Factored Moment Resistance **13—21.5**

13—21.5.1 **General**

The factored moment resistance of the composite section shall be computed by the method of transformed sections. E for the concrete shall be taken as $5000\sqrt{f_c'}$. E for wood shall be taken as E_{50} in accordance with Table 13—11.2(a) and E for steel shall be taken as 200 000 MPa.

13—21.5.2 **Factored Positive Moment Resistance**

The provisions of Clause 13—21.5.1 shall be considered satisfied if the factored positive moment resistance is taken as:

$$M_u = \phi f_{bu} S k_d k_m k_s$$

where: ϕ is in accordance with Table 13—4.4 for dimension lumber
f_{bu} is in accordance with Table 13—11.2(a)
S_b is in accordance with Clause 13—21.5.2.1
k_d is in accordance with Clause 13—5.3
k_m is in accordance with Clause 13—5.6
k_s is in accordance with Table 13—6.2.1

13—21.5.2.1 **Section Modulus**

The elastic section modulus S_b of the composite section shall be obtained by transforming the concrete into an equivalent area of wood. Only the net section of the wood base shall be considered, ignoring the capacity of the spliced butt joint.

13—21.5.3 **Factored Negative Moment Resistance**

The provisions of Clause 13—21.5.1 shall be considered satisfied if the factored negative moment resistance is taken as the smaller of:

$$M_u = \phi f_{bu} S_b k_d k_m k_s$$

and

$$M_u = \phi_s f_y \frac{E_{50} S_t}{E_s}$$

where: ϕ, f_{bu}, k_d, k_m, and k_s are in accordance with Clause 13—21.5.2,
S_b and S_t are in accordance with Clause 13—21.5.3.1,
ϕ_s is the performance factor for steel in tension in accordance with the provisions of Section 10,
f_y is the yield strength of the steel in accordance with the provisions of Section 10.

13—21.5.3.1 **Section Modulus**

The elastic section moduli, S_b and S_t, of the composite section shall be obtained by transforming the steel into an equivalent area of wood. Where the provisions of Clause 13—21.4.2(b) apply, only that portion of the wood below the bottom of the grooves shall be used.

Prestressed Wood Decks 13—22

General **13—22.1**

The provisions of this clause apply to longitudinally-laminated wood decks which are transversely prestressed.

Prestressing Materials **13—22.2**

13—22.2.1 **Prestressing Steel**

Prestressing steel shall be high-strength bars satisfying the requirements of uncoated high-strength bars in CSA Standard G279-M1982, *Steel for Prestressed Concrete Tendons*.

13—22.2.2 **Anchorages**

Anchorages shall be capable of developing 95% of the ultimate strength of the bars. After tensioning and seating, anchorages shall sustain applied loads without slippage, distortion, or other changes which would contribute to prestress loss. The dimensions of the anchorages shall be established by the manufacturer of the prestressing system. Details of the anchorage distribution bulkhead shall be in accordance with Clause 13—22.4.

13—22.2.3 **Couplers**

Couplers shall develop at least the ultimate strength of the uncoated tendons.

13—22.2.4 **Stress Limitations**

The stress in the prestressing steel shall not exceed f_{py} nor $0.85f_{pu}$ at jacking, and $0.80f_{pu}$ at transfer.

Where coating or galvanizing of the bar reduces the anchorage or coupler capacity, these maximum values shall be correspondingly reduced.

Design of Prestressing System **13 — 22.3**

The prestressing system may be external or internal as shown in Figures 13 — 22.4.4.1(a) and 13 — 22.4.4.1(b), respectively.

13 — 22.3.1 **Steel/Wood Ratio**

The steel/wood ratio shall not exceed 0.0016.

13 — 22.3.2 **Distributed Normal Pressure on Laminations**

The maximum value of the normal pressure N_i, at jacking, assumed as uniformly distributed over an area s × b, shall be $0.25 f_{q\ell}$, where $f_{q\ell}$ is in accordance with Table 13 — 22.3.2.

When stressing is in accordance with Clause 13 — 22.3.3, the minimum final assumed uniformly-distributed pressure N_f shall be taken as $0.5 N_i$. In no case shall N_f be less than 0.35 MPa.

13 — 22.3.3 **Stressing Procedure**

The stressing procedure shall be that specified in Table 13 — 22.3.3.

able **Limiting Pressure Perpendicular to Grain $f_{q\ell}$, MPa** **13 — 22.3.2**

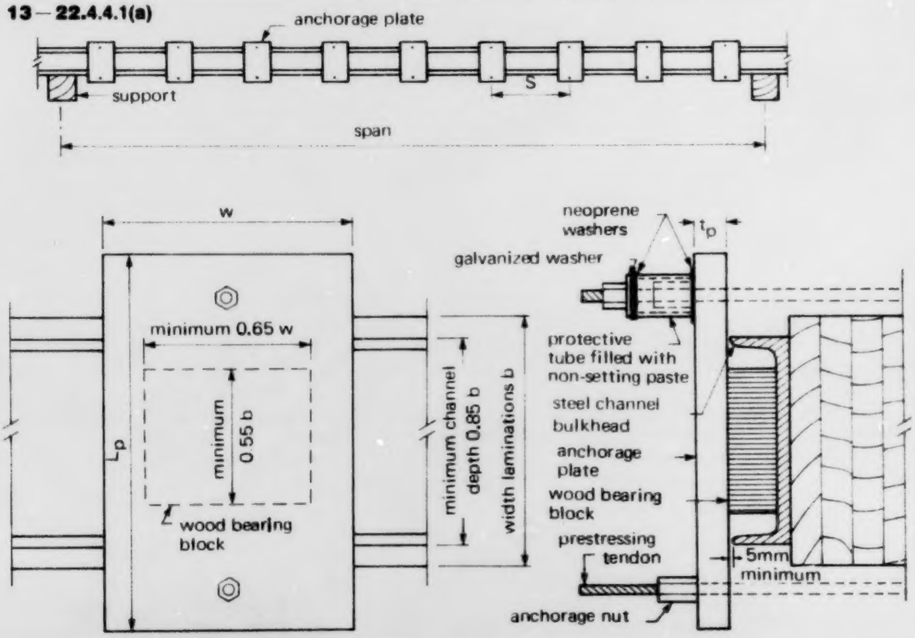
Individual Species or Species Combination	$f_{q\ell}$
Douglas Fir, Larch	5.0
Hem-Fir	4.4
Spruce-Pine-Fir	3.3
Lodgepole Pine	3.3
Jack Pine	5.0
Red Pine	3.3
White Pine	3.3

able **Stressing Procedure** **13 — 22.3.3**

Prestress Stage	
1	Stress to 100% of N_i
2	Restress to 100% of N_i about 1 week after completion of Stage 1.
3*	Restress to 100% of N_i 4 to 8 weeks after completion of Stage 2.

* For existing bridges which are more than 10 years old, Stage 3 may be performed up to 6 months after Stage 2.

igure **External Prestressing System** **13 — 22.4.4.1(a)**


anchorage plate
support
s
span
w
minimum 0.65 w
minimum 0.55 b
L_p
wood bearing block
minimum channel depth 0.85 b
width laminations b
neoprene washers
t_p
galvanized washer
protective tube filled with non-setting paste
steel channel bulkhead
anchorage plate
wood bearing block
prestressing tendon
5mm minimum
anchorage nut

Design of Distribution Bulkhead **13 — 22.4**

13 — 22.4.1 **General**

Prestressed wood decks shall incorporate continuous distribution bulkheads along the sides of the deck. These bulkheads may be discontinuous over supports only and shall be capable of effectively distributing the prestress forces to the wood.

13 — 22.4.2 **Factored Bearing Resistance To Prestress**

The factored resistance of the wood in bearing shall be taken as:

$$R_u = \phi f_{q\ell} A_b$$

where: ϕ is in accordance with Table 13 — 4.4,
$f_{q\ell}$ is in accordance with Table 13 — 22.3.2,
A_b is in accordance with Clause 13 — 22.4.3.

The load factor for prestress bearing shall be taken as the maximum specified in Section 2 for secondary prestressing effects.

13 — 22.4.3 **Bearing Area for Prestress Force**

The direct bearing area A_b for the prestress force at each station shall be determined by the designer of the distribution bulkhead system. Where the bulkhead satisfies the requirements of Clause 13 — 22.4.4, the bearing area shall be taken as:

$$A_b = b_b \times L_b$$

where b_b and L_b are in accordance with the provisions of Clause 13 — 22.4.4.4.

13 — 22.4.4 **Steel Channel Bulkhead**

The provisions of Clause 13 — 22.4.1 shall be considered satisfied if the distribution bulkhead consists of steel channel and steel anchorage plates in accordance with the provisions of Clauses 13 — 22.4.4.1 to 13 — 22.4.4.5.

13 — 22.4.4.1 **Channel**

The depth of the steel channel bulkheads, in accordance with Figure 13 — 22.4.4.1(a) and (b), shall be at least 85% of the depth of the edge lamination, but shall in no case exceed the depth of the wood. The minimum section properties of the channel shall be in accordance with Table 13 — 22.4.4.1.

Figure **Internal Prestressing System** **13 — 22.4.4.1(b)**

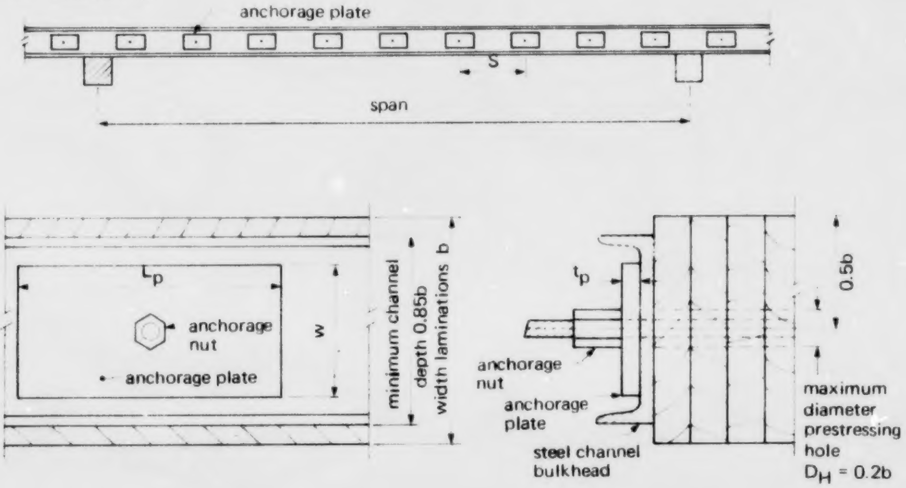


Table **Minimum Section Properties, Steel Channel Bulkheads** **13 — 22.4.4.1**

Depth of Laminated Deck, mm	Moment of Inertia* of Channel, mm^4	Web Thickness, mm
184	550 000	9.5
235	1 000 000	11.0
286	1 400 000	11.0

* Moment of inertia about the minor axis.

13—22.4.4.2 **Anchorage Plate**

The ratio of the length L_p of the anchorage plate to the width w of the plate in accordance with Figure 13—22.4.4.1(a) and (b) shall be such that:

$$1.0 \le \frac{L_p}{w} \le 2.0$$

The thickness of the anchorage plate t_p shall be not less than:

$$\frac{L_p}{12}$$

13—22.4.4.3 **Spacing of Prestressing Stations**

The spacings between stations shall not exceed 6 times the depth of the wood deck. In no case shall the spacing exceed 1500 mm nor be less than $22D_h$.

13—22.4.4.4 **Effective Bearing Area**

The width b_b of the direct bearing area on the edge lamination shall be taken as the height of the steel channel. The length L_b of the direct bearing area along the channel shall be taken as follows.

a) **Internal System** — The length L_p of the anchorage plate plus twice the thickness of the web of the channel.

b) **External System** — The width w of the anchorage plate plus twice the width of the flange of the channel.

13—22.4.4.5 **Web Stiffening**

In external systems, web stiffening shall be provided to the steel channel, beneath the anchorage plate, to transmit prestress forces simultaneously to the flanges and web of the channel. This requirement may be considered satisfied if the following method is employed:

A wood bearing block is provided in accordance with Figure 13—22.4.4.1(a) with the grain oriented parallel to the applied load. The cross-sectional dimensions of the block shall not be less than 55% of the depth of the deck nor less than 65% of the width w of the anchorage plate. The thickness of the block shall be such that it protrudes at least 5 mm beyond the flanges. The bearing pressure on the wood block shall not exceed f_{pu} in accordance with Table 13—11.2(b).

Laminated Deck **13—22.5**

13—22.5.1 **General**

The provisions of Clause 13—21.2.1.2, related to butt joint stagger only, shall apply to all prestressed vertically-laminated decks.

13—22.5.2 **Lamination Dimensions**

Decking shall consist of longitudinally-laminated dimension lumber 38 to 76 mm thick with widths no less than 190 mm.

13—22.5.3 **Holes in Laminations for Internal Systems**

The diameter of prestressing holes drilled in laminated decking for an internal system shall not exceed 20% of the width of the lamination.

13—22.5.4 **Nailing**

Each lamination shall be fastened by common nails to the preceding one at approximately 1 m intervals. The nails shall be driven alternately near the top and bottom edges of the laminations. The nails shall be of sufficient length to pass through two laminations and at least halfway through the third, except that in no case shall the nails exceed 152 mm in length.

13—22.5.5 **Support Anchorage**

Decks shall be provided with a tie-down system which can be installed and/or engaged after Stage 2 prestressing, as specified in Table 13—22.3.3, has been performed. During assembly, prestressed wood decks shall not be anchored to the supports.

Factored Moment Resistance **13—22.6**

The factored moment resistance of prestressed wood decks shall be taken as:

$$M_u = \phi f_{bu} S k_d k_m k_s$$

where:
ϕ is in accordance with Table 13—4.4 for dimension lumber,
f_{bu} is in accordance with Table 13—11.2(a),
S is in accordance with Clause 13—22.6.1,
k_d is in accordance with Clause 13—5.3,
k_m is in accordance with Table 13—5.6,
k_s is in accordance with Table 13—6.2.1.

13—22.6.1 **Net Section**

In computing the factored moment resistance, the net section of the deck shall be considered. The net section shall be taken at a transverse line of butt joints, ignoring the effects of prestressing holes.

Factored Transverse Shear Resistance **13 — 22.7**

Prestressed wood decks shall be designed to resist the effects of vertical shear in the transverse direction by friction between the laminations.

The factored resistance in transverse shear of prestressed wood decks shall be taken as:

$$V_{tu} = \phi N_t d \mu$$

where ϕ and μ are in accordance with Table 13 — 22.7 and N_t is in accordance with Clause 13 — 22.3.2.

Table **Friction, Prestressed Decks** **13 — 22.7**

Performance Factor	ϕ	0.9
Friction Coefficient	μ	0.35 Dressed Material 0.45 Rough Sawn

Environmental Protection **13 — 22.8**

13 — 22.8.1 **Hardware**

All steel components of the prestressing system shall be in accordance with Clause 13 — 15. In addition to surface treatment, all anchorages and bars of external systems shall be protected by a system equivalent to that shown in Figure 13 — 22.8.1. The protective tubing shall be sealed against moisture penetration using neoprene seals and shall be provided with a collapsible connection to facilitate movement during prestressing.

13 — 22.8.2 **Preservative Treatment for Deck**

The decking shall be preservative treated in accordance with Clauses 13 — 16.1, 13 — 16.4, and 13 — 16.5.

Design Details **13 — 22.9**

13 — 22.9.1 **Curbs and Guiderails**

The curbs and guiderails shall not be connected directly to the steel distribution bulkhead. The anchorage of the curbs and guiderails shall be made directly to the prestressed wood deck.

13 — 22.9.2 **Prestress Failure Protection**

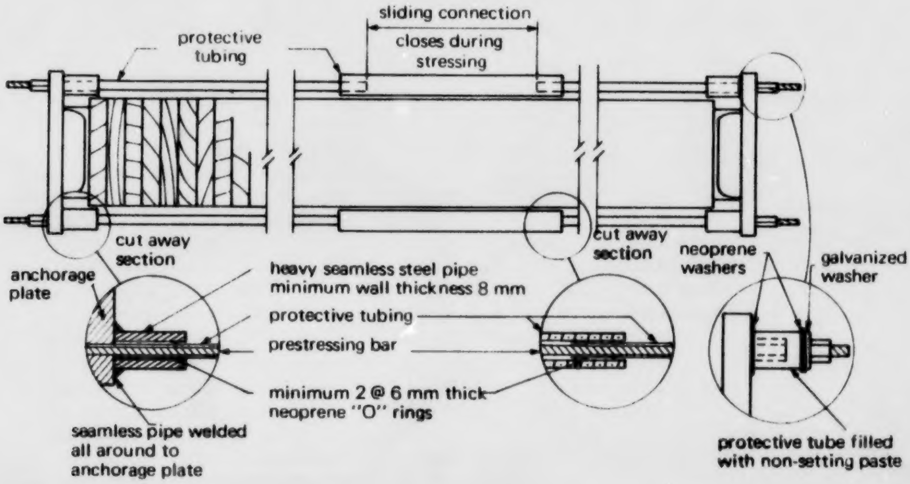
To guard against possible failure of a prestressing bar after construction, the steel distribution bulkhead shall be designed to restrain all prestressing components. This protection shall be removable so that the prestressing anchorages are accessible.

Wearing Surface **13 — 23**

General **13 — 23.1**

A wearing surface of untreated wood, plant-mix asphalt, asphalt planks, tar and chips, concrete or other approved material shall be used on all wood bridges except for those in wood-concrete composite construction.

Figure **Protection for External System** **13 — 22.8.1**


sliding connection
closes during stressing
protective tubing
cut away section
anchorage plate
heavy seamless steel pipe minimum wall thickness 8 mm
protective tubing
prestressing bar
minimum 2 @ 6 mm thick neoprene "O" rings
seamless pipe welded all around to anchorage plate
cut away section
neoprene washers
galvanized washer
protective tube filled with non-setting paste

Plant-Mix Asphalt **13—23.2**

Plant-mix bituminous wearing surfaces shall have a minimum thickness of 50 mm. The asphalt shall be reinforced by stapling to the deck a 76 mm × 152 mm 10/10 welded wire mesh conforming to CSA Standard G30.5, *Welded Steel Wire Fabric for Concrete Reinforcement*, using 50 mm long staples. The 76 mm spacing of the mesh shall be placed perpendicular to the laminations. The mesh shall be at a distance of 15 mm above the wood surface. Reinforcement of the asphalt is not required for prestressed wood decks.

Drainage **13—24**

General **13—24.1**

Positive drainage paths shall be provided to ensure drainage away from all parts of the structure. Moisture traps shall be avoided.

Requirements for Deck **13—24.2**

The crown and crossfall of the deck shall be in accordance with the provisions of Section 1. A minimum vertical gap of 150 mm, interrupted as appropriate, shall be provided between the deck and the curb along the length of the bridge.

Table **Minimum Values of the Effective Length Factor k** **A13—1**

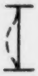
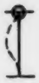
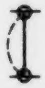
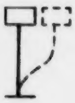
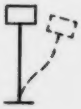
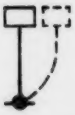
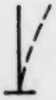
End Restraint	Minimum Effective Length Factor k	
Effectively held in position and restrained against rotation at both ends.	0.65	
Effectively held in position at both ends, restrained against rotation at one end.	0.80	
Effectively held in position at both ends, but not restrained against rotation.	1.00	
Effectively held in position and restrained against rotation at one end and, at the other, restrained against rotation but not held in position.	1.10	
Effectively held in position and restrained against rotation at one end and, at the other, partially restrained against rotation but not held in position.	1.50	
Effectively held in position at one end but not restrained against rotation and, at the other end, restrained against rotation but not held in position.	2.00	
Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end.	2.00	

Table **A, S, and e_o for Piles at 0.55 of the Effective Length Below the Butt** **A13 — 2**

D_b/D_t	k_u	0.7	0.6	0.5	0.4	0.3
356	A	7.81	8.13	8.45	8.71	9.03
×	S	3.10	3.28	3.44	3.62	3.82
254 mm	e_o	79	62	47	36	29
356	A	7.35	7.68	8.06	8.45	8.77
×	S	2.82	3.02	3.23	3.44	3.67
229 mm	e_o	77	60	46	35	29
356	A	6.90	7.29	7.68	8.13	8.58
×	S	2.56	2.79	3.02	3.28	3.54
203 mm	e_o	74	58	44	35	28
330	A	7.10	7.29	7.48	7.68	7.94
×	S	2.67	2.79	2.90	3.02	3.15
254 mm	e_o	76	59	45	35	28
330	A	6.65	6.90	7.16	7.42	7.68
×	S	2.41	2.56	2.70	2.85	3.02
229 mm	e_o	73	57	44	34	28
330	A	6.19	6.52	6.84	7.16	7.48
×	S	2.18	2.34	2.52	2.70	2.90
203 mm	e_o	70	55	42	33	27
330	A	5.74	6.13	6.52	6.90	7.29
×	S	1.95	2.15	2.34	2.56	2.79
178 mm	e_o	67	53	41	32	27
305	A	5.94	6.13	6.32	6.52	6.71
×	S	2.05	2.15	2.25	2.34	2.44
229 mm	e_o	70	54	42	33	27
305	A	5.48	5.74	6.00	6.26	6.52
×	S	1.84	1.95	2.08	2.21	2.34
203 mm	e_o	67	52	41	32	26
305	A	5.16	5.42	5.68	6.00	6.32
×	S	1.64	1.77	1.92	2.08	2.25
178 mm	e_o	64	50	39	31	26
279	A	4.90	5.10	5.23	5.42	5.61
×	S	1.52	1.61	1.69	1.79	1.87
203 mm	e_o	64	50	39	31	26
279	A	4.52	4.71	4.97	5.16	5.42
×	S	1.36	1.46	1.56	1.67	1.77
178 mm	e_o	61	48	37	30	25
279	A	4.16	4.41	4.67	4.95	5.24
×	S	1.20	1.31	1.43	1.56	1.69
152 mm	e_o	58	46	36	29	25
254	A	3.61	3.81	3.99	4.21	4.41
×	S	0.97	1.05	1.13	1.22	1.31
152 mm	e_o	55	43	34	28	24
229	A	3.11	3.25	3.37	3.52	3.61
×	S	0.77	0.82	0.87	0.93	0.99
152 mm	e_o	52	41	33	27	23

A — $mm^2 \times 10^4$ S — $mm^3 \times 10^6$ e_o — mm

Table **A13—3** **η to be Used in Computing P_{cr} for Piles**

D_b (mm)	D_t (mm)	k_u				
		0.3	0.4	0.5	0.6	0.7
356	254	8.25	7.74	7.25	6.77	6.31
	229	7.87	7.25	6.66	6.09	5.55
	203	7.49	6.78	6.09	5.45	4.79
330	254	8.55	8.13	7.72	7.33	6.94
	229	8.13	7.59	7.07	6.56	6.08
	203	7.72	7.07	6.44	5.84	5.27
	178	7.33	6.56	5.84	5.18	4.52
305	229	8.44	7.99	7.55	7.13	6.71
	203	8.00	7.41	6.85	6.31	5.80
	178	7.56	6.85	6.18	5.55	4.95
279	203	8.33	7.84	7.37	6.91	6.47
	178	7.84	7.21	6.61	6.03	5.49
	152	7.36	6.61	5.89	5.22	4.59
254	152	7.64	6.96	6.32	5.70	5.12
229	152	7.99	7.41	6.85	6.32	5.80

14 Evaluation of Existing Bridges

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Reference Publications

Section 14 refers to the following publications:

CSA Standards

CAN3-A23.2M-77	*Methods of Test for Concrete*
G40.20M-1978	*General Requirements for Rolled or Welded Structural Quality Steel*

Revised Statutes of Ontario

Highway Traffic Act of Ontario, 1979

Scope **14—1**

The provisions of this section pertain to the evaluation and posting of existing bridges for the ultimate limit states. Guidelines for evaluations including condition survey, analysis, and load testing of bridges are provided. Provisions for the calculation of single and multiple posting limits are given.

Provisions for the evaluation of soil-steel structures and masonry bridges are not given.

Definitions **14—2**

Controlled Vehicle — a vehicle having known axle weights and dimensions which, when crossing a structure, complies with controls imposed by the structure owner.

Condition Survey — a survey carried out by an engineer in order to ascertain the inherent features and conditions of all the components and their assemblages in a structure.

Delamination — a fracture plane below the surface of concrete or other material.

Deterioration — the loss or impairment of structural material in a component.

Distress — excessive cracking or deformation.

Non-Destructive Testing — the *in situ* determination of the physical properties of a component without impairment of, or removal of any material, excluding the necessary surface preparation for testing.

Posting — signing of a bridge for load restrictions according to the provision of the Highway Traffic Act of Ontario.

Routine Inspection — the examination of bridge conditions according to prescribed procedures and at intervals determined by the owner in order to determine the safety and serviceability of the bridge.

Scale-Down Factor — the fraction of the evaluation load that a bridge can safely sustain.

Secondary Components — components which play a minor role in the overall performance of the structure.

Spalling — separation and removal of a portion of the surface concrete.

Transitory Loads — as given in Section 2, Table 2—5.1(a).

Notation **14—3**

A_r	the normal area of rivets, mm²
A_s	the area of tensile reinforcing steel per unit width, mm²/m
B_r	the factored bearing resistance of a member or component, N
d	the effective depth of a section, mm; or the diameter of a rivet, mm
e	the edge distance, mm
F	a scale-down factor
F_u	the specified minimum tensile strength of structural steel, MPa
F_y	the specified minimum yield stress, yield point, or yield strength of structural steel, MPa
f'_c	the compressive strength of concrete, MPa
f_{pu}	the specified strength of prestressing steel, MPa
f_y	the specified yield stress of reinforcing steel, MPa
L_o	the factored wheel load including dynamic load allowance
m	the number of faying surfaces or shear planes in a riveted joint equal to 1 for rivets in single shear, and 2 for rivets in double shear
n	the number of rivets
q	the average tensile reinforcement percentage at the midspan of the panel slab in the two directions in which the steel is placed
R_n	the unfactored resistance of a component, kN
T_f	the tensile force in a member or component at the ultimate limit state, N
T_r	the factored tensile resistance of a riveted joint, N
t	the thickness of a deck slab, mm; or the thickness of steel component, mm
V_f	the shear force at the ultimate limit state, N
V_r	the factored shear resistance of a riveted joint, N
α_L	the live-load factor
ϕ	the performance factor
ϕ_r	the performance factor for rivet steel

Need for Evaluation **14—4**

The need for an evaluation shall be based on any of the following.

a) Routine inspection revealing impairment due to deterioration, distress, or damage.
b) Changes in design and evaluation load specifications, road classification, or load restriction.
c) Rehabilitation affecting load-carrying capacity.
d) An application for a permit to allow a controlled vehicle to use the bridge.

Condition Survey

14 — 5

14 — 5.1 **General**

Inspections shall be carried out to the satisfaction of the engineer responsible for the evaluation of the bridge. The engineer shall determine whether the bridge is in general conformance with available plans. When the plans are not available or are found to be inaccurate, all pertinent components shall be measured and data recorded.

Changes in geometry and alignment of members, especially compression members, shall be investigated and recorded.

Any distress and deterioration shall be recorded. Records shall be documented so that they are available for comparison with future condition surveys.

The mechanical properties of the materials which influence the load-carrying capacity of the bridge shall be determined in accordance with Clauses 14 — 5.2, 14 — 5.3 and 14 — 5.4.

The moduli of elasticity of the materials shall be as specified in the appropriate Code sections, or based on tests.

14 — 5.2 **Steel and Iron**

14 — 5.2.1 **Inspection of Components**

The extent and location of deterioration, cracking and other distress shall be determined.

14 — 5.2.2 **Strength Properties**

a) **Structural Steel**

The yield stress of steel shall be determined by one of the following:

i) The specified yield stress as given on the plans.

ii) Mill certificates; however, the yield stress used shall not exceed the guaranteed minimum as specified for the grade of steel.

iii) Coupon testing using test specimens obtained and tested in accordance with CSA Standard G40.20M, *General Requirements for Rolled or Welded Structural Quality Steel*. Test coupons shall be taken from parts of components where the stress is known to be low and preferably from the thicker sections of those parts.

One test coupon shall be taken from every fifty tonnes of steel in the bridge with a minimum of three coupons per bridge. Test results of coupons that differ from the average by 15% or more shall not be acceptable, and additional test coupons shall be obtained when necessary.

The yield stress of an individual member shall be taken as 90% of the test value of a coupon taken from the same piece of steel being analysed.

The yield stress used for evaluation shall be taken as 90% of the average of the acceptable test coupons taken from representative portions of the bridge.

iv) If plans, mill certificates, or coupon tests are not available, the values shown in Table 14 — 5.2.2 shall be used.

b) **Wrought Iron**

The yield stress of wrought iron shall be taken as 180 MPa.

Concrete

14 — 5.3

14 — 5.3.1 **Inspection of Components**

The locations and extent of spalling, delamination, cracks and any other distress shall be determined.

The extent of exposed or deteriorated prestressed and non-prestressed reinforcement shall be determined.

14 — 5.3.2 **Strength Properties**

a) **Concrete**

The strength of concrete shall be determined in accordance with methods given in Table 14 — 5.3.2(a). Both ultrasonic pulse velocity and surface hardness methods may be considered as acceptable non-destructive tests. When the strength of the concrete is to be determined from drilled cores, the tests shall be in accordance with CSA Standard CAN3-A23.2M, *Methods of Tests for Concrete*. A minimum of two cores per span or, in the case of a single-span bridge, a minimum of four cores, shall be taken.

b) **Reinforcing Steel**

The specified yield stress of reinforcing steel given on the plans shall be used. If specified yield stress or test results are unavailable, the values given in Table 14 — 5.3.2(b). shall be used.

c) **Prestressing Steel**

The specified strength of prestressing steel given on the plans shall be used. If specified strength or test results are unavailable, the following values shall be used for steel strand and wire:

- for bridges built before 1963 — f_{pu} = 1600 MPa
- for bridges built after 1962 — f_{pu} = 1725 MPa

Table **Properties of Steel When Plans, Mill Certificates or Coupon Tests Are Unavailable** **14 — 5.2.2**

Date of Bridge Construction	Specified Minimum Yield Stress F_y to be Assumed, MPa	Specified Minimum Tensile Strength F_u to be Assumed, MPa	
	Structural Steel	Rivet Steel	Structural Steel
Before 1905	180	340	360
1905 — 1932	210	340	420
1933 — 1975	230	340	420
After 1975	250	340	420

Note — Silicon steel and nickel steel with specified minimum yield stresses higher than 250 MPa were available after 1961.

Table **Methods of Determining the Compressive Strength of Concrete, f'_c** **14 — 5.3.2(a)**

	Compressive Strength of Concrete f'_c	
Construction Specification	No Visible Sign of Concrete Damage	Visible Sign of Concrete Damage
Available	The compressive strength of concrete as specified on the plans or from tests of drilled cores having a diameter of not less than 100 mm.	The compressive strength of the sound concrete as specified on the plans or from tests of drilled cores. Where appropriate, small-diameter cores from damaged areas may be used to determine the type and extent of deterioration. Nondestructive test methods may be used to correlate the concrete strength of the damaged zone with that of the sound concrete in the same bridge.
Not Available	20 MPa or from tests of drilled cores having a diameter of not less than 100 mm.	For sound concrete, 20 MPa or from tests of drilled cores having a diameter of not less than 100 mm. Where appropriate, small-diameter cores from damaged areas may be used to determine the type and extent of deterioration. Nondestructive test methods may be used to correlate the concrete strength of the damaged zones with that of the sound concrete in the same bridge.

Table **Properties of Reinforcing Steel When Plans or Test Results are Unavailable** **14 — 5.3.2(b)**

Date of Bridge Construction	Specified Minimum Yield Stress, F_y, to be Assumed, MPa			
	Unknown Grade	Structural Grade	Intermediate Grade	Hard Grade
Before 1914	210			
1914 — 1955	230	230	275	345
1956 — 1978	275	275	345	415
After 1978	350	300	350	400

Wood **14 — 5.4**

14 — 5.4.1 **Inspection of Components**

The location and extent of deterioration and distress due to wear, length of end split, weathering, chemical attack, fungus, insect and vermin attack, and fire damage shall be determined.

14 — 5.4.2 **Strength Properties**

For evaluation of sound components, the specified strengths as given in Clause 13 — 11.2 shall be used. Where the species of wood is unknown, the specified strength shall be assumed to be that of Spruce-Pine-Fir species. An increase in the specified strength shall be justified by having the species identified through laboratory testing and visual identification.

Bearings **14 — 5.5**

14 — 5.5.1 **Inspection of Bearings**

Evidence of horizontal and vertical movements and any misalignments relative to supported members shall be determined. The extent and type of deterioration, cracking, and other distress shall be determined.

Evaluation Procedures **14 — 6**

General **14 — 6.1**

The evaluation of existing bridges shall be carried out using all of the appropriate sections of this Code for dead loads, Ontario bridge evaluation loads, and controlled vehicle loads. Evaluations for other transitory load effects may be considered if deemed to be of consequence by the engineer.

Components and details subject to significant fatigue effects shall be identified and inspected. Evaluation for fatigue may be considered if deemed to be of consequence by the engineer.

The effects of secondary components may be considered in accordance with the appropriate sections of this Code.

Limit States **14 — 6.2**

Ultimate limit states shall be used in evaluation. Evaluation for fatigue, where deemed to be of consequence by the engineer, shall be for serviceability limit state, Type I.

Loading Combinations **14 — 6.3**

Evaluations shall be carried out for ULS Loading Combination 1 of Section 2, Table 2 — 5.1(a). ULS Loading Combinations 2 and 3 of that table may be considered if they are deemed to be of consequence by the engineer.

The live load factor may be reduced as specified in Section 2, Clause 2 — 5.4.3, if the bridge is to be re-evaluated within 5 years.

For controlled vehicles, the provisions of Section 2, Clause 2 — 4.3.2.10 for restricted speed, and Clause 2 — 5.4.2 for load and modification factors shall be used.

The provisions of Section 2, Clause 2 — 6.1.2, shall apply to evaluation for Levels 1, 2 and 3 loads.

Effects of Deterioration **14 — 6.4**

Consideration shall be given to the reduction of load-carrying capacity of a component due to loss of material. Where deterioration in a component is extensive and affects the load distribution characteristics of the structure, the following shall also be considered:

a) increases in load effects in adjacent components, and
b) decreases in load effects in the deteriorated components.

Factored Resistance **14 — 6.5**

14 — 6.5.1 **General**

Factored resistance of reinforced concrete, prestressed concrete, structural steel, wood, and substructure members shall be in accordance with their respective Code sections.

Factored resistance of concrete deck slabs, steel trusses, rivets, and bearings shall be in accordance with Clause 14 — 6.5.

14 — 6.5.2 **Concrete Deck Slabs**

The factored resistance of concrete deck slabs which are solid, of constant thickness, and supported by girders, beams, or other linear supports, shall be determined for the maximum wheel load as specified in Section 2. The analysis shall be done in accordance with the provisions of Section 7. The unfactored resistance of both composite and non-composite concrete deck slabs may be determined from the graphs given in Figure A14 — 1(a) through A14 — 1(e) provided that the following conditions are satisfied.

a) The span length of a slab panel does not exceed 4.5 m. For span lengths which are perpendicular to the direction of traffic, the slab extends at least 1.0 m

beyond the centreline of the external longitudinal supports of a panel. In the case of an external panel, a curb integral with the slab is considered equivalent to the 1.0 m overhang provided that the combined cross-sectional area of slab and curb, beyond the centreline of the external girder, is not less than the cross-sectional area of 1.0 m overhang of deck slab.
The span length is parallel to the main reinforcement and is determined in accordance with Section 7.

b) The span-to-thickness ratio of the slab does not exceed 20.

c) The minimum slab thickness of sound concrete is not less than 150 mm. For slabs of variable thickness, the minimum slab thickness shall be used.

d) All cross frames or diaphragms extend throughout the cross-section of the bridge between external girders, and the maximum spacing of such cross frames or diaphragms is determined in accordance with Section 7.

e) Edge stiffening is in accordance with Section 7.

The value of q used in conjunction with Figures A14 — 1(a) to A14 — 1(e) is taken as the average tensile reinforcement percentage at the midspan of the slab, in the two directions in which the reinforcement is placed. The value of d is taken as the average of the effective depths at the midspan of the slab, in the two directions in which the reinforcement is placed.

14 — 6.5.3 Steel Trusses

The factored resistance of steel truss components shall be established in accordance with the provisions of Section 10. When a truss fails to meet the design requirements of Section 10, its strength analysis shall be carried out using rational methods.

14 — 6.5.4 Rivets

a) Rivets in Tension

The factored tensile resistance, T_r, of rivets in tension shall be taken as:

$$T_r = \phi_r n A_r F_u$$

where: $\phi_r = 0.67$

Rivets shall be adequate to resist the sum of the external load and any additional tensile load caused by deformation of the connected parts.

b) Rivets in Shear

The factored shear resistance of a riveted joint subject to shear, V_r, shall be taken as the lesser of the following:

i) Factored Bearing Resistance, B_r, given by:

$$B_r = \phi\, tne\, F_u \leq 3\phi\, tnd\, F_u, \text{ and}$$

ii) Factored Shear Resistance, V_r, given by:

$$V_r = 0.75\phi_r nm A_r F_u$$

where F_u is the smaller of the ultimate strengths of the contacting materials, ϕ_r = 0.67 and ϕ = 0.90.

c) Rivets in Shear and Tension

A rivet which is required to develop resistance to a tensile force and shear force which results from loads at the ultimate limit state shall satisfy the following relationship:

$$V_f^2 + 0.56T_f^2 \leq 0.56\phi_r^2(A_r F_u)^2$$

14 — 6.5.5 Bearings

Bearings failing to meet the provisions of Section 4 shall be evaluated using rational methods, which are subject to approval. Forces induced by frozen bearings and the effects of freeing the bearings shall be considered.

Bridge Posting 14 — 7

General 14 — 7.1

The calculation for establishing the posting limits for bridges shall be carried out in accordance with Clause 14 — 7.2. In certain cases a single posting load, based upon engineering judgement and experience, may be used, subject to approval.

A concrete bridge shall not be posted for restricted loads if it has been carrying normal traffic without signs of distress. Such a bridge shall be inspected at intervals recommended by the evaluation engineer.

Scale-Down Factor, F 14 — 7.2

14 — 7.2.1 Evaluation Levels 1, 2, 3 and Controlled Vehicles

For Evaluation Levels 1, 2, and 3, and the controlled vehicle, the smallest value of F for the appropriate load case shall be calculated from the following equation for all structural components of the bridge except deck slabs:

$$F = \frac{\phi R_n - \text{load effects due to factored dead load}}{\text{load effects due to factored live load}}$$

14—7.2.2 **Concrete Deck Slabs**

For wheel loads of Levels 1, 2 and 3, and controlled vehicles, the scale-down Factors F for decks shall be calculated from the following equation:

$$F = \frac{\phi R_n}{L_o}$$

where R_n is obtained in accordance with Clause 14—6.5.2.
For Levels 1, 2, 3 and controlled vehicle, $\phi = 0.5$.

Calculations for Posting Loads **14—7.3**

For Evaluation Levels 1, 2 and 3 the smallest values of F from Clause 14—7.2 shall be calculated and applied as follows.

a) **Level 3**
  - $F \geq 1$ — posting is not required.

b) **Levels 2 or 3**
  - $F > 0.3$ — triple posting is required with the posting loads for Levels 1, 2 and 3 obtained from Figures 14—7.3(a), 14—7.3(b) and 14—7.3(c), respectively.
  - $F < 0.3$ — single posting is required with the posting load for Level 1 obtained from Figure 14—7.3(a).

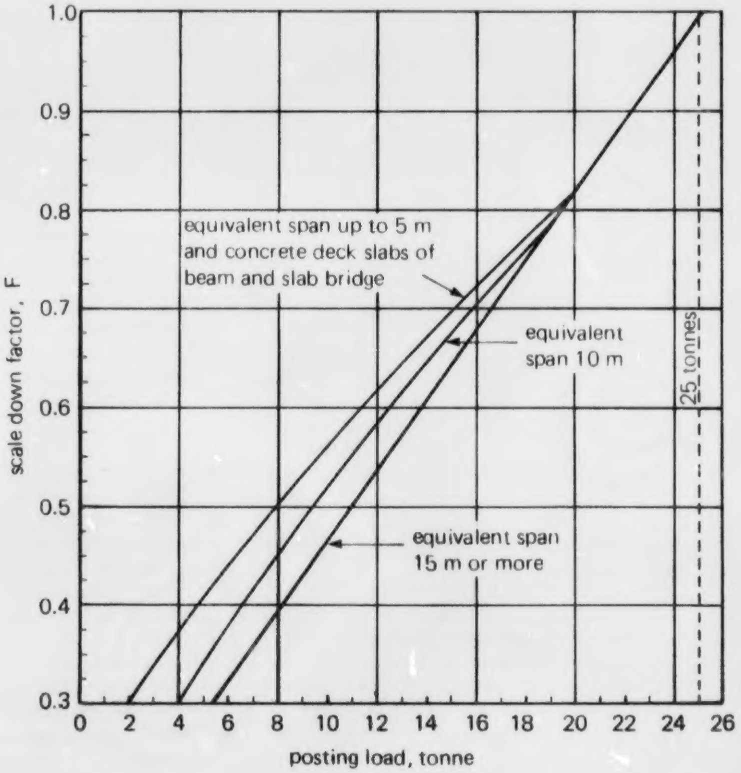
c) **Level 1**
  - $F < 0.3$ — consideration shall be given to closing the bridge.

For single and multiple spans the equivalent span length to be used in determining the posting loads from Figure 14—7.3(a), (b) and (c) shall be based upon the loaded length of the influence line for the load effect under consideration. That part of the influence line which is removed by one span from the section under consideration shall be ignored.

For positive moments and shears, the equivalent span shall be based upon the span in which the section under consideration is located.

For negative moments and reactions, the equivalent span shall be based upon the span in which the section is located plus the adjacent span which is loaded.

Figure **Posting Loads Corresponding to Evaluation Level 1 Loads** **14—7.3(a)**



Calculations for Controlled Vehicles **14 — 7.4**

14 — 7.4.1 For controlled vehicles the smallest value of F shall be obtained from Clause 14 — 7.2. When this value of F is larger than 1, the bridge shall be considered to be safe for the vehicle.

Posting Signs **14 — 7.5**

14 — 7.5.1 **Single Posting Sign**

The posting sign shall be in accordance with the Highway Traffic Act of Ontario and show the maximum gross vehicle weight to the nearest tonne which is permitted for any highway vehicle on the bridge.

14 — 7.5.2 **Triple Posting Sign**

The posting sign shall be in accordance with the Highway Traffic Act of Ontario and

Figure **Posting Loads Corresponding to Evaluation Level 2 Loads** **14 — 7.3(b)**

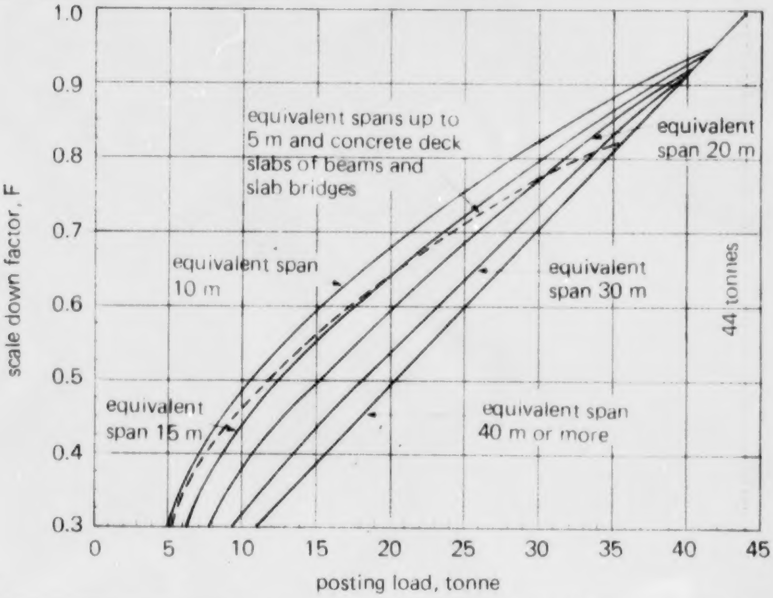
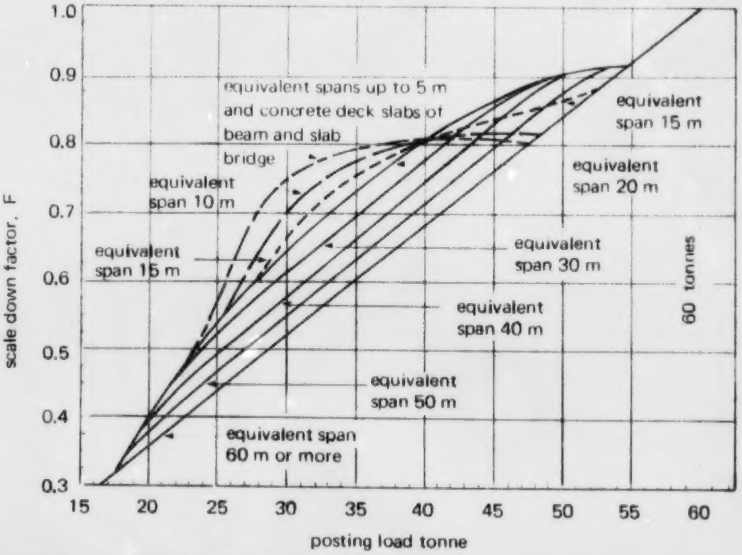


Figure **Posting Loads Corresponding to Evaluation Level 3 Loads** **14 — 7.3(c)**



show the following three vehicle types, from top to bottom, respectively, and the maximum gross vehicle weight to the nearest tonne permitted on the bridge for each type.

a) Single unit vehicle corresponding to Evaluation Level 1 loads.
b) Single unit vehicle and single trailer combination corresponding to Evaluation Level 2 loads.
c) Single unit vehicle and more than one trailer combination corresponding to Evaluation Level 3 loads.

Load Testing **14—8**

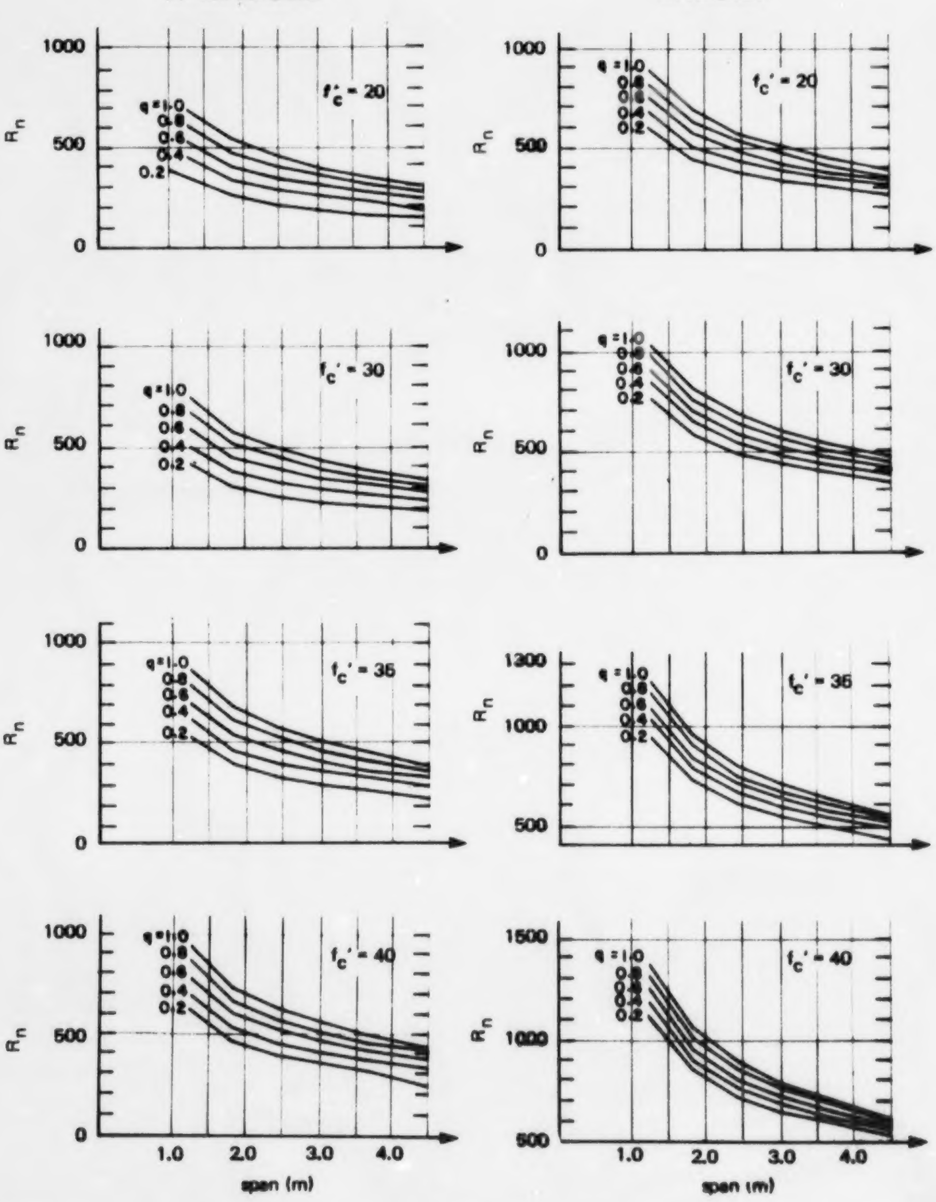
Bridges shall be considered for load testing if analytical evaluations are deemed to be unsatisfactory. When a load test is proposed as part of the evaluation procedure, such a test, including details of loads, instrumentation, condition survey and analysis, is subject to approval.

Appendix **Load-Carrying Capacity of Concrete Deck Slabs** **A14 — 1**

The unfactored resistance of composite and non-composite deck slabs, R_n, is given in Figures A14 — 1(a) to A14 — 1(e).

For a deck thickness other than those of Figures A14 — 1(a) to A14 — 1(e), the value of R_n shall be obtained by linear interpolation between the values for the deck thicknesses next above and below it.

Figure **R_n of 150 mm Concrete Deck** **A14 — 1(a)**


(a) non-composite

(b) composite

Figure **R_n of 175 mm Concrete Deck** **A14 — 1(b)**

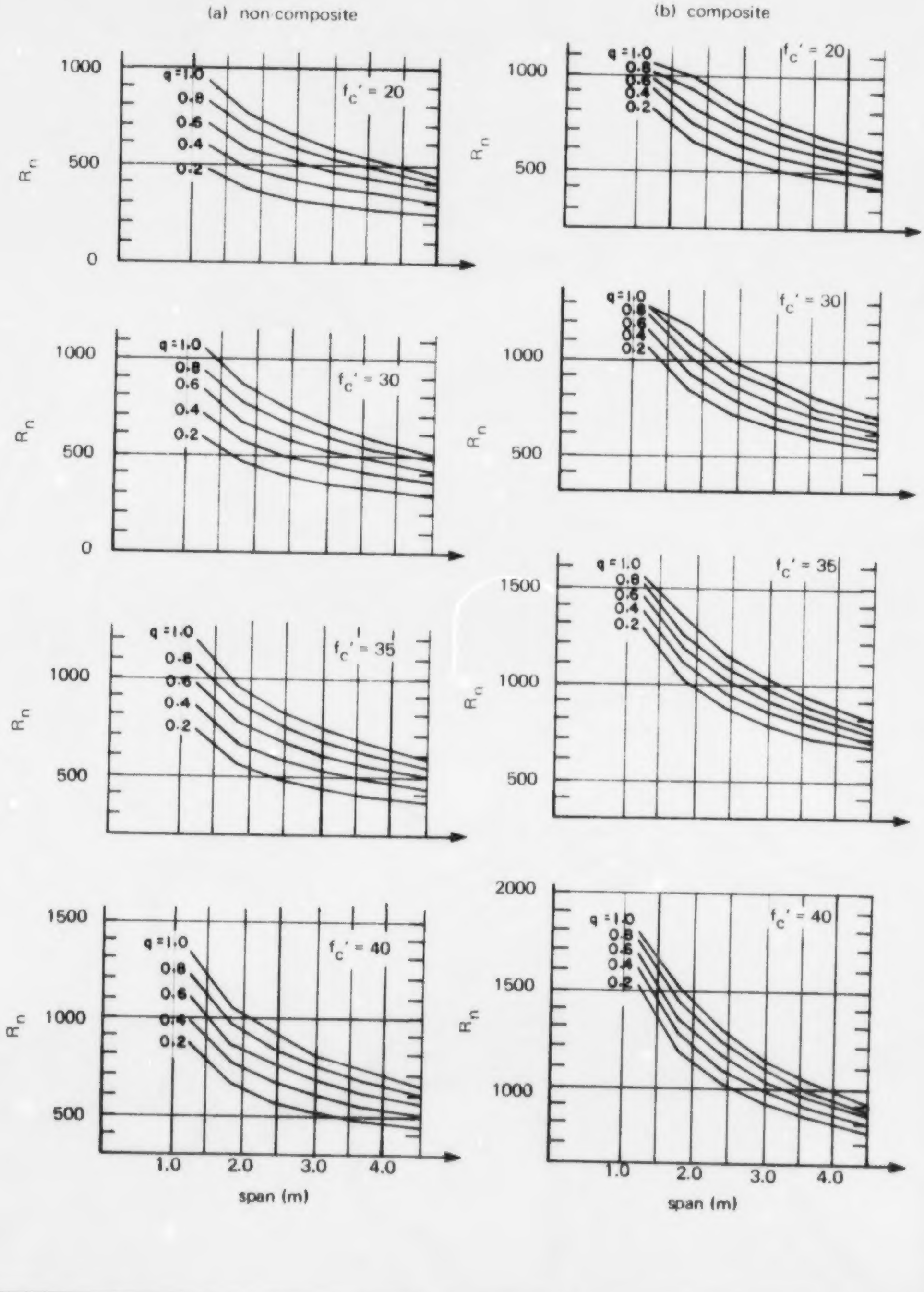
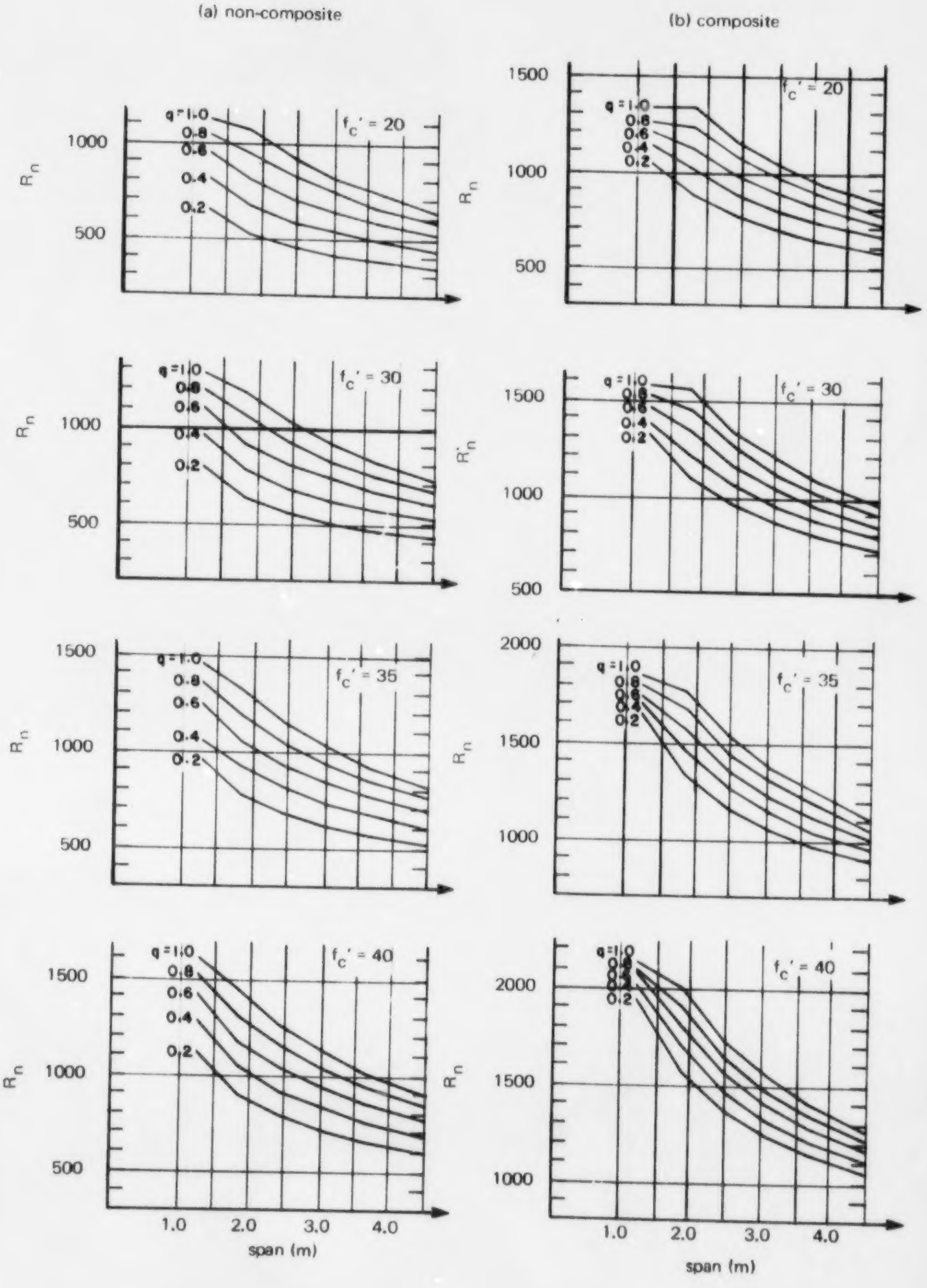


Figure **R_n of 200 mm Concrete Deck** **A14 — 1(c)**



(a) non-composite

(b) composite

Figure **R_n of 225 mm Concrete Deck** **A14 — 1(d)**

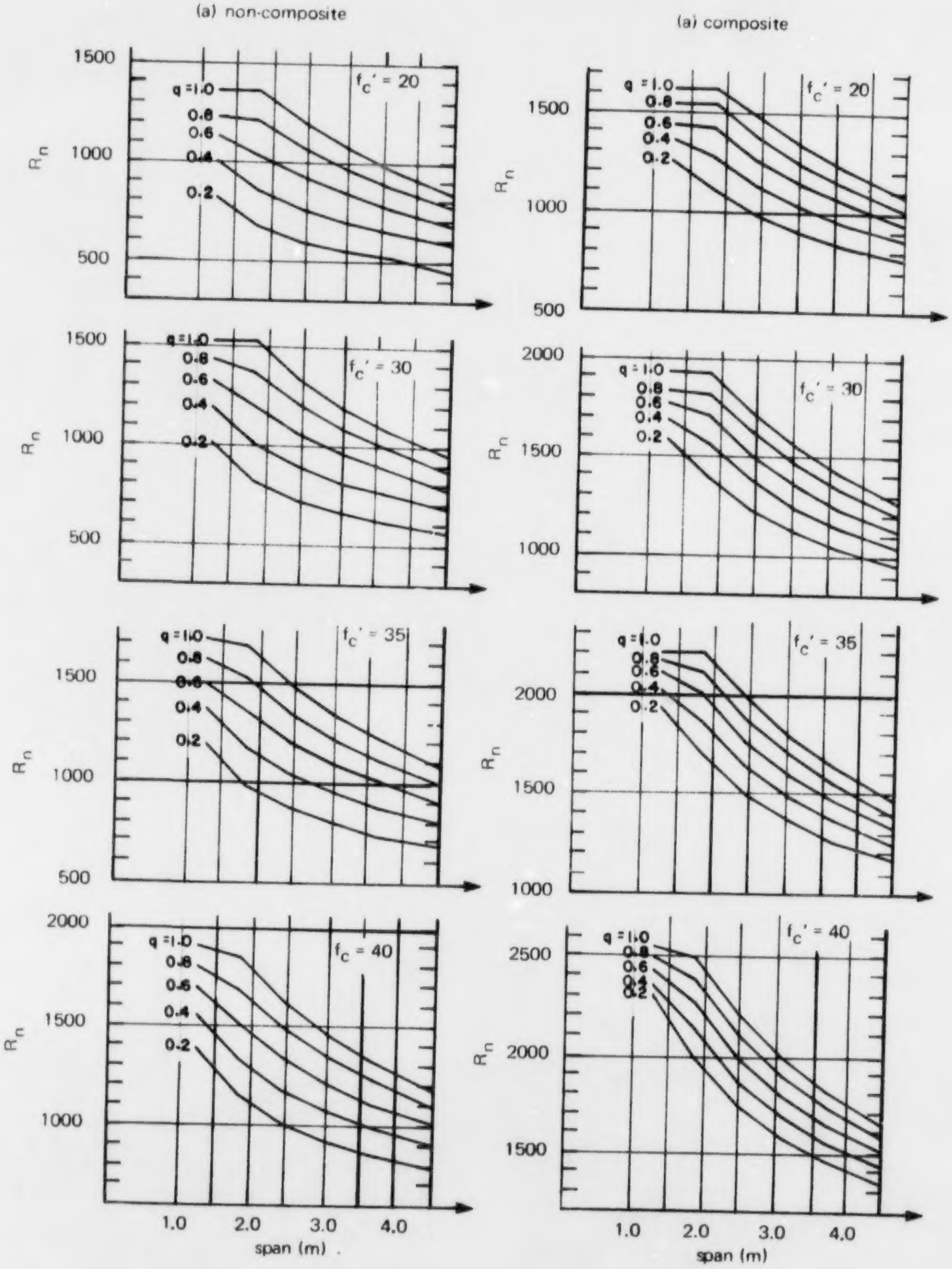
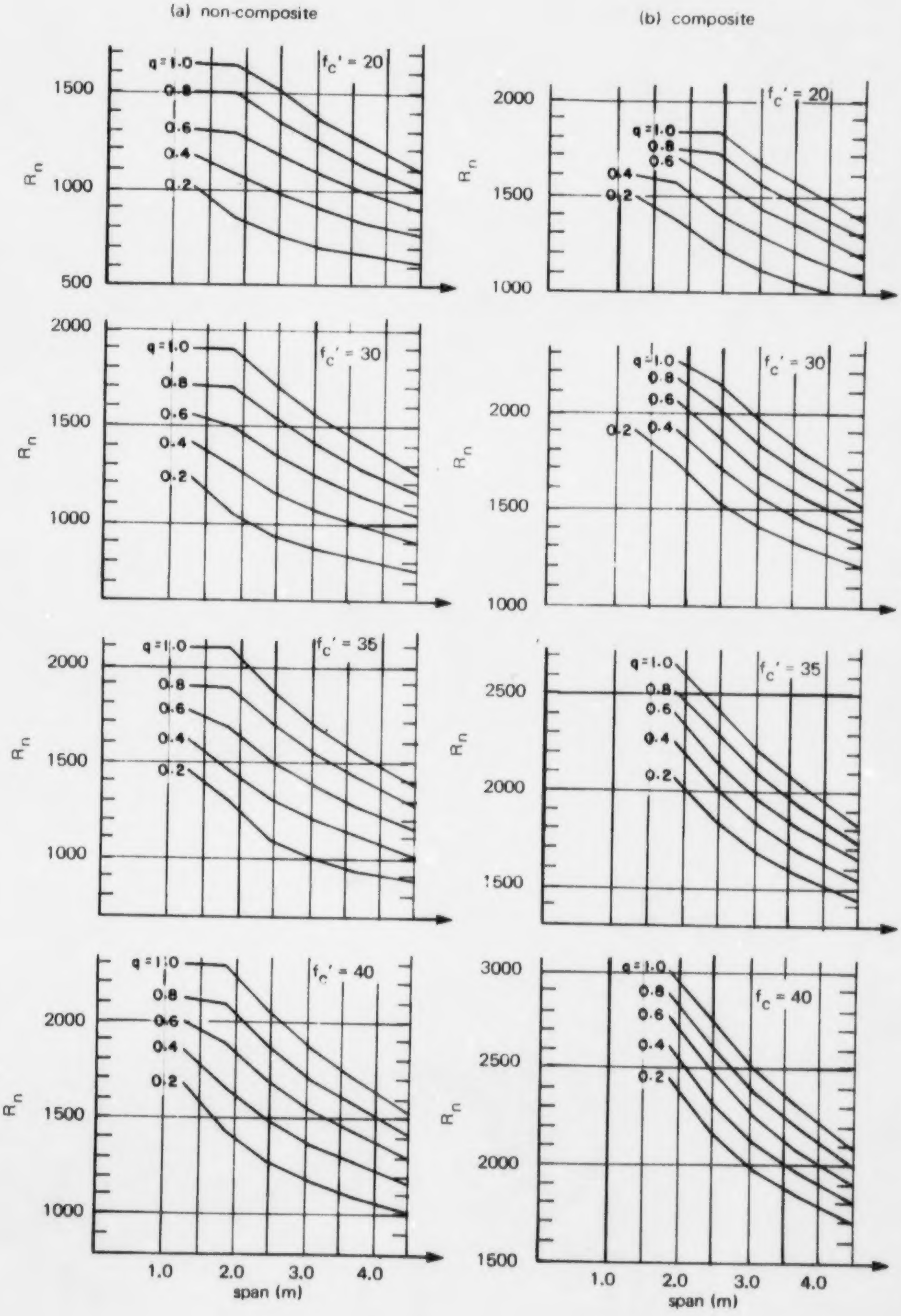


Figure **R_n of 250 mm Concrete Deck** **A14—1(e)**



(a) non-composite

(b) composite

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Preface to Second Edition

The First Edition of the Ontario Highway Bridge Design Code was issued in 1979, with P.F. Csagoly as Chairman of the Code Development Committee. It was published by the Ontario Ministry of Transportation and Communications for use in the Province of Ontario, and was amended by the subsequent issuing of three Addenda, in September 1979, December 1980 and May 1982. The Code was written in SI units and limit states format and, because of the many changes in approach compared to codes previously used in Ontario, it was issued on a trial basis. It has been applied to the design and evaluation of virtually all bridges on the King's Highways system since 1979, and has also been used in most of the larger municipal jurisdictions.

The Second Edition represents an extensive rewrite of the Code and Commentary based on four years experience of application of the First Edition and new research and development information. The number of sections has been reduced from 17 to 14 by combining previous Sections 2, 3, 4 and 17 into the new Section 2. The following changes in section numbers have also been made: Section 5 becomes Section 3, Section 15 becomes Section 4, and Section 16 becomes Section 5. While modifications have been made to all sections, the most significant changes appear in Sections 2, 3, 6 and 13.

Compliance with this Second Edition of the Code will become a requirement for the design and evaluation of all bridges on the public road system in Ontario at a date to be announced by the Minister. The Commentary will remain an advisory document.

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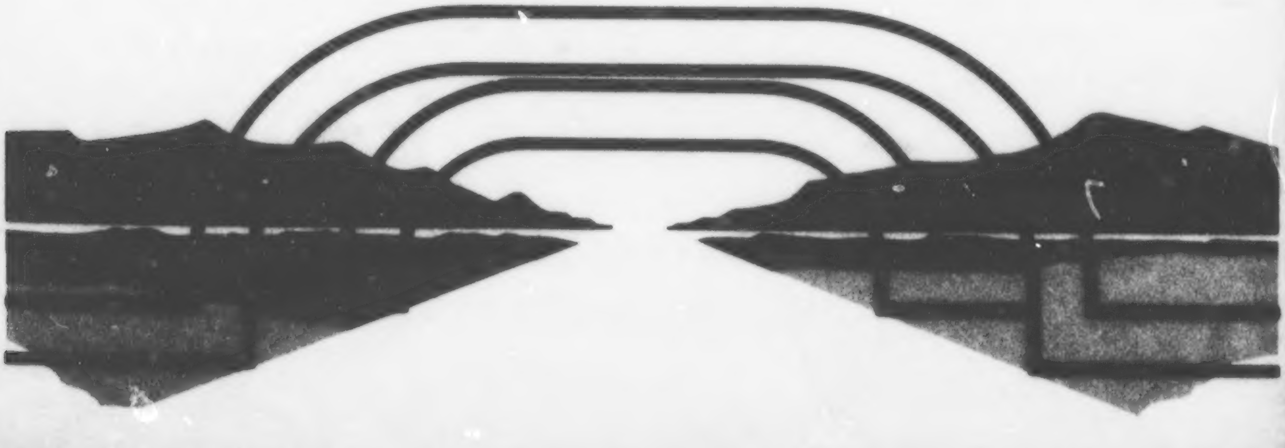


C0 Overall Requirements

Clause Contents

C0.1 Scope of Code
C0.3 General Provisions
C0.4 Limit States Method
C0.5 Design of New Bridges
C0.6 Evaluation of Existing Bridges
C0.7 Rehabilitation Design of Existing Bridges
C0.8 Construction
C0.9 Application of Code
References

Scope of Code

C0 — 1 The individual span limits of 125 m will cover nearly all the bridges in Ontario. The few bridges with longer spans which currently exist are generally international or interprovincial bridges which are excluded due to dual jurisdiction. Cable-supported structures generally have spans in excess of 125 m and are excluded for this reason. Masonry bridges are excluded as they are no longer built, and special provisions requiring approval will be needed for the evaluation or rehabilitation of those which exist. Background information on the preparation of the First Edition of the Code is given in Reference 1.

General Provisions

C0 — 3

C0 — 3.1 This information is essential in order to carry out an evaluation of the bridge when this becomes necessary at a later date.

C0 — 3.2 It has been normal practice in the past for the designer to stamp the plans. This clause now requires the designer and the checker to stamp and sign the plans, thus certifying that the design was done in accordance with the Code. Having a design and independent check is considered normal good practice.

C0 — 3.3 This Code has been written with the expectation that the "General Conditions, Standard Specifications and Supplemental Specifications" of the Ministry will apply for construction. If other construction specifications are to be used, they require prior approval to ensure that they are consistent with Code provisions.

C0 — 3.4 This clause aims to reduce the probability of failures during construction by requiring that all construction drawings be prepared under the supervision of a professional engineer.

C0 — 3.5 This requires the engineer to consider the safety of the structure during construction and the effect of built-in stresses as a consequence of the erection sequence. Should the constructor have different proposals, he must consult with the engineer regarding changes in the construction method, and obtain approval for any changes.

C0 — 3.6 This clause recognizes the possibility of variations and developments in bridge design not covered by the Code and indicates the need for approval for such departures.

C0 — 3.7 The design should not only meet the technical requirements of the Code, but should result in an economical and aesthetically pleasing structure. Economic considerations should include construction costs, design costs, and lifetime maintenance costs. The needs of fabrication and erection should be considered at the design stage.

C0 — 3.8 The requirement for ease of access for inspection can only be ensured at the design stage. Ease of inspection is essential for the long-term maintenance and durability of bridges, and the Code requires access to the interior of box girders and between the ends of the superstructure and the abutment wall. Non-structural features that may impair the durability or safety of the bridge should be avoided. To improve durability and minimize maintenance costs, it is important for the engineer to provide self-protecting features. Important among these are the avoidance of ponding of water or accumulation of debris on the structure, and keeping the number of deck joints to a practical minimum. Past performance of deck joints has not generally been satisfactory, and leakage has caused high maintenance costs. It is thus desirable to design continuous structures with as few deck joints as possible.

C0 — 3.9 This statement reflects the engineer's responsibility to the wider community to seek to preserve the environment to the greatest possible extent. Important features are the appearance of the bridge itself and its effect on the hydrology of the surrounding area.

Limit States Methods

C0 — 4

C0 — 4.1 This clause is of fundamental importance and follows directly from the adoption of a limit states philosophy for the Code. The serviceability limit states should be treated as equally important as the ultimate limit states for new structures. While the usual limit states have been identified in the Code, the engineer is responsible for ascertaining that all possible limit states are properly investigated, particularly if dealing with unusual structures or conditions.

In the calibration of this Code [2, 3] the reliability is measured by a safety index, β, illustrated in Figure C0 — 4.1. The safety index is defined as the ratio of the mean to the standard deviation of random variable X which is a measure of structural safety.

For ultimate limit states, the variable X is taken as the natural logarithm of R/S where S is the load effect and R is the corresponding resistance.

The target value of safety index for ultimate limit states is selected as 3.5 for the projected bridge lifetime of 50 years. This choice is based on a statistical analysis of a selection of existing bridges in Ontario [3] considered representative of present and future construction.

For serviceability limit states Type II, the variable X is taken as the safety margin, that is, $X = R - S$. For this limit state the target value of the safety index is selected as 0 for the appropriate period.

The process described [2, 3] was applied to the superstructures of steel, reinforced concrete, and prestressed concrete bridges. Wood bridges were calibrated later in a similar way, and exhibited a somewhat larger deviation from the target safety index value. This calibration process did not cover substructures, or soil-steel structures, which were calibrated against previous performance.

C0 — 4.2

Elastic methods of analysis are prescribed for determining the response of the structure at the ultimate limit states, this being in keeping with current North American codes such as AASHTO, ACI and NBC and most European codes. While it is true that the structure is unlikely to behave elastically throughout the load range and that plastic redistribution will normally occur before an ultimate limit state is reached, it is recognized that suitable inelastic methods of analysis are not generally available to the designer at the present time. Inelastic methods require approval for use unless specifically allowed by the Code — as is the case, for example, with yield line theory for slabs or with model testing.

Design of New Bridges **C0 — 5**

The Code has been written with the design of new bridges as the expected major application. Section 14 addresses evaluation of existing bridges and does not apply to the design of new bridges. Any new bridge certified as meeting the requirements of the Code shall be in accordance with the Overall Requirements, Sections 1 to 13 inclusive, and the Appendices.

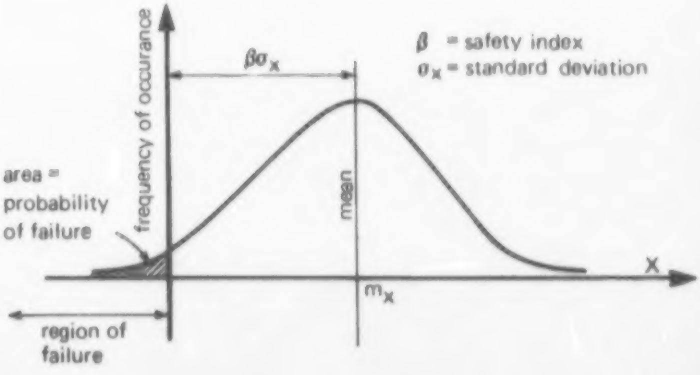
Evaluation of Existing Bridges **C0 — 6**

Section 14 prescribes clauses for the evaluation of existing bridges, some of which are modifications of new bridge clauses and others are additional. It is expected that the other sections of the Code will also be applied to the evaluation of existing bridges. As the new bridge clauses apply to a 50-year life, the future service life of an existing bridge needs to be assessed in selecting appropriate factors for evaluation.

Rehabilitation Design of Existing Bridges **C0 — 7**

Bridges being rehabilitated now and in the near future will have been designed to the AASHTO or CSA specifications, not this Code, and will generally have an anticipated future service life of less than 50 years. If these bridges are to be rehabilitated economically, it may not be possible to strengthen all members to full Code level. In some cases, strengthening to some agreed posting level may be adequate. In other cases, replacement of deteriorated members to restore original strength may be the best answer.

Figure **Safety Index** **C0 — 4.1**



The Code does not, at present, cover all bridge rehabilitation possibilities, but should be used for evaluation and load posting. The preferred strengthening or rehabilitation approach is for the engineer to decide, but the design criteria are to be submitted for approval.

Construction **C0 — 8**

The Code covers calculations at various states of construction, but more calibration work remains to be done on safety levels for construction. In the past, safety levels for construction have been reduced below those required for completed bridges, on the grounds that the construction condition is for a short time and the bridge is not open to the public. Failures during construction have been more frequent than failures of completed bridges, however, and the previous philosophy has been re-examined for this Code. Because the loading during construction is less well defined it may not always be safe to reduce construction safety levels below the permanent condition levels.

Application of Code **C0 — 9**

C0 — 9.1 Design criteria submitted for approval for structures outside the Code limits should have the same target safety index of $\beta = 3.5$ for the ultimate limit states.

The ASCE recommended design live load for bridges [4] agrees quite well with this Code at 125 m span, and is the preferred loading for spans in excess of 125 m.

C0 — 9.2 The Code does not cover any of the mechanical or electrical aspects of movable bridge operation. However, the structural aspects of movable highway bridges, when locked in a static position, should meet the requirements of this Code.

C0 — 9.3 The railing loading provisions of the Code indicate good safety practice. This clause allows an owner to specify and accept responsibility for a lower safety level if he considers that special conditions warrant such a change.

C0 — 9.4 Model testing should be carried out by an engineer with extensive experience in order to provide a sound prediction of prototype behaviour from model results.

The full-scale load testing of bridges for evaluation has been carried out for a number of years by the Ministry. It is Ministry policy to ensure that such testing is carried out under the control of an experienced engineer in view of the fact that testing, especially proof loading, can involve the risk of bridge failure and possible injury or loss of life.

C0 — 9.5 This statement is intended to discourage use of single-load path structures, defined as that category of structures in which failure of a single component causes collapse of the bridge.

The Code does not, at present, list all single-load path structural types, and the term may be subject to varied interpretation. Considerable discretion is thus given to the engineer, but he should submit his proposed structure for approval if there is doubt as to whether it is a single-load path or multi-load path structure.

References

[1] *Ontario Highway Bridge Design Code 1979*, Volume II, "Supplement to Preface," SP, Ministry of Transportation and Communications (MTC), Downsview, Ontario, 1979.

[2] Lind, N.C. and Nowak, A.S., "Calculation of Load and Performance Factors," Report submitted to MTC, University of Waterloo, Waterloo, Ontario, May 1978.

[3] Nowak, A.S. and Agarwal, A.C., "Calibration of the Ontario Highway Bridge Design Code," MTC, Downsview, Ontario, 1979.

[4] "Recommended Design Loads for Bridges," ASCE Committee on Loads and Forces on Bridges. ASCE, Journal of the Structural Division, Proc. Paper 16404, July, 1981.

C1 General Features

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Scope

C1 — 1

The scope of Section 1 is somewhat broader than that of equivalent sections of AASHTO [7] and CSA-S6 [6]. More coverage is given to certain aspects of design that have been the source of problems in the past. The chief of these is the hydraulic design, which exerts a considerable influence over the cost, safety and maintenance of river structures. Scour and undermining of bridges and open footing culverts, the cause of many failures, are the subject of detailed provisions, and special attention is paid to the prevention of uplift failures of corrugated steel culverts.

Definitions

C1 — 2

Definitions have been abstracted from existing glossaries wherever possible, but have been modified where necessary to suit the purpose of the Code. Principal references are the RTAC *Guide to Bridge Hydraulics* [1] and the ASCE *Nomenclature for Hydraulics* [2].

Bridge Deck Criteria

C1 — 4

Bridge Deck Cross-Section

C1 — 4.1

C1 — 4.1.1.2

Single-Lane Bridges

Single-lane bridges are undesirable unless the traffic volume is exceptionally low. Five metres is the narrowest deck width which will permit use of the normal range of snowplow equipment, and allow a passenger vehicle to pass a disabled commercial vehicle.

C1 — 4.1.2

C1 — 4.1.2.1

Bridge Deck Shoulder Widths

Prior to development of the New Jersey type concrete barrier wall with sloping safety profile, most bridge railings were provided with a 600 mm wide safety curb. "Short" bridges were provided with a full-width shoulder, i.e., the inside of the barrier wall or railing lined up with the edge of the shoulder on the approach roadway. "Long" bridges were provided with a narrow shoulder, usually 1.5 m wide including the curb.

Development of the safety profile barrier rendered the safety curb undesirable and, at the same time, prompted a re-evaluation of bridge width criteria. The new bridge width criteria adopted in 1978 and given in this clause are based on the following considerations.

a) The minimum clear shoulder width of 0.91 m permitted on bridges under the previous criteria has proven to be satisfactory over the years. A value of 1.0 m is therefore considered adequate for the minimum metric shoulder width.
b) By making the bridge shoulder width a function of the roadway shoulder width, it was related to traffic requirements, i.e., design speed and lane width.
c) The width of travelled way is the same as previously provided for "short" bridges.
d) A comprehensive cost analysis of all bridges on the provincial highway system built in a five-year period had indicated that there would have been no increase in the overall cost if the new criteria had been used.

C1 — 4.1.2.2

Continuous curbs are usually provided on urban roads. In these cases it is desirable that there be no change in the curb line at bridges.

C1 — 4.1.2.3

Since the speed-change lane is available for evasive action to the motorist using the main driving lane, the shoulder width of 1.0 m is considered to be adequate.

C1 — 4.1.2.4

It is sometimes necessary to increase shoulder widths on bridges located on horizontal curves in order to maintain required stopping sight distances. The shoulder widths should not exceed 3.0 m, as the traffic would tend to use the wider shoulders as additional driving lanes.

C1 — 4.1.2.5

This provision is to ensure that when future sidewalks or bikeways are added they will meet the requirements of Clause 1 — 4.1.3.

C1 — 4.1.2.6

Shoulder Widths for Short and Long Bridges

For short bridges, the extra cost of providing full-width shoulders can sometimes be low enough to warrant aligning the bridge rail with the approach guiderail. For long bridges, the reduction in cost may warrant the use of narrower shoulder widths.

C1 — 4.1.3

Sidewalk and Bikeway Widths

The provisions of this clause permit the bridge sidewalk to line up with the normal range of sidewalk widths on the approaches. Alignment of the bridge curb with the roadway curb is considered desirable for safety reasons. The 1.5 m requirement is based on the minimum acceptable sidewalk width for two people.

The 1.0 m minimum width per lane of bikeways agrees closely with the 1.07 m recommended by AASHTO [3]. The 0.25 m minimum clearance from curbs or other raised objects represents a compromise between various values given by AASHTO; these are 0.15 m minimum and 0.3 m desirable clearances from curbs, and 0.15 m minimum and 0.6 m desirable clearances from other raised objects.

C1 — 4.1.4 **Width of Bridge Deck Median**

Continuation of the roadway median width over a bridge is desirable for traffic using the high-speed or passing lanes.

Curbs **C1 — 4.2**

Curbs should not be built in conjunction with safety profile barrier walls as they interfere with the proper interaction of vehicle and barrier wall, and they can cause vehicles to vault over the wall.

Sidewalk Height and Crossfall **C1 — 4.3**

The height of the sidewalk should be matched to that of the adjacent roadway curb for the sake of safety, appearance, and uniformity. The height of 150 mm, used by the Ministry for many years, has been found adequate. The sidewalk should be sloped 2% to permit proper drainage.

Roadway Superelevation and Crossfall **C1 — 4.4**

Superelevation should normally be limited to 6% to prevent sideslip of vehicles at low speed on any icy road surface. The minimum slope of 2% is for drainage.

Roadway Surface **C1 — 4.5**

Concrete bridge deck roadway surfaces should be transversely grooved or broom finished to improve skid resistance. Asphalt surfaces usually provide adequate skid resistance without special treatment. Limestone aggregate should not be used for concrete deck surfaces where the approach road surface is asphalt.

Longitudinal Deck Profile **C1 — 4.6**

Discontinuities in the longitudinal profile can be minimized by the following methods.

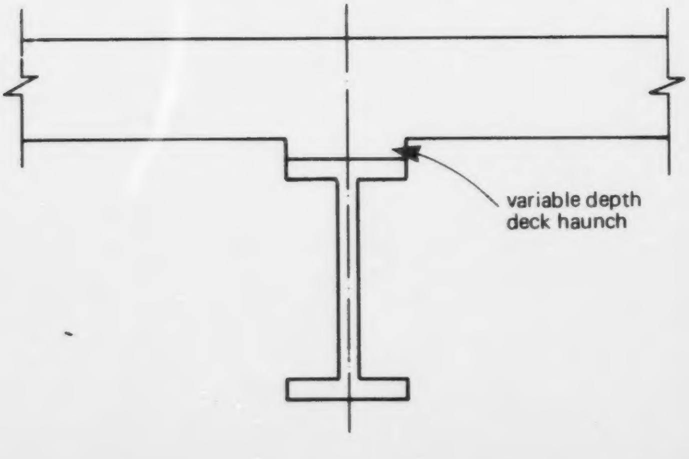
a) Cambering bridge beams and girders to take account of dead load deflection.
b) Providing concrete deck haunches above the top flanges of bridge girders to allow for camber caused by welding and fabricating procedures, or caused by prestressing from the time prestressed girders are made to the time of concrete deck placement. A typical haunch detail is shown in Figure C1 — 4.6.
c) Placing the top surfaces of expansion joints parallel to the adjacent roadway surface rather than horizontal.

Bridge Deck Drainage **C1 — 4.7**

C1 — 4.7.1 General

Prevention of flow onto bridge decks is necessary to avoid ice formation due to rapid cooling of the deck.

Figure **Typical Concrete Deck Haunch Detail** **C1 — 4.6**



C1 — 4.7.2 Size and Number of Drains

The size and spacing of deck drains is normally controlled by the permissible encroachment of water onto the traffic lanes. Most provincial highway bridges have shoulders sloping toward the edge of the bridge deck; this normally provides adequate width for gutter flow without the need to encroach much on the traffic lanes. If there is no shoulder, or if the shoulder slopes toward the travelled way, the specified maximum encroachment of 1.5 m should leave sufficient room for safe travel within the curb lane. Higher drainage standards are used for bridge decks than for the approach roadway, because of the greater hazards for vehicles which lose control on bridges. A procedure for determining deck drain spacing is contained in Chapter E of the MTC *Drainage Manual* [4]. Details of deck drains are given in the MTC *Structural Manual* [5]. Deck drains have caused significant problems in the past, and should be avoided wherever possible.

C1 — 4.7.3 Bridges Without Deck Drains

Bridges with a small to medium deck area may not require deck drains. In such cases, the water may be allowed to run to the end of the bridge where it can be carried away by catchbasins or other means. For railway overheads or other structures where deck drains are not usually permitted, catchbasins should be provided at both ends of the structure.

C1 — 4.7.4 Downspouts and Downpipe Systems

C1 — 4.7.4.1 Location and Length of Downspouts

Improper location of deck drains and inadequate length of downspouts are major causes of local deterioration of structural steel and concrete bridges. The 45° spread of water discharged by the downspouts is intended to take account of wind action.

C1 — 4.7.4.2 Projection Below Superstructure

The length of downspouts projecting below the superstructure should be minimized for the sake of appearance.

C1 — 4.7.4.3 Downpipe Systems

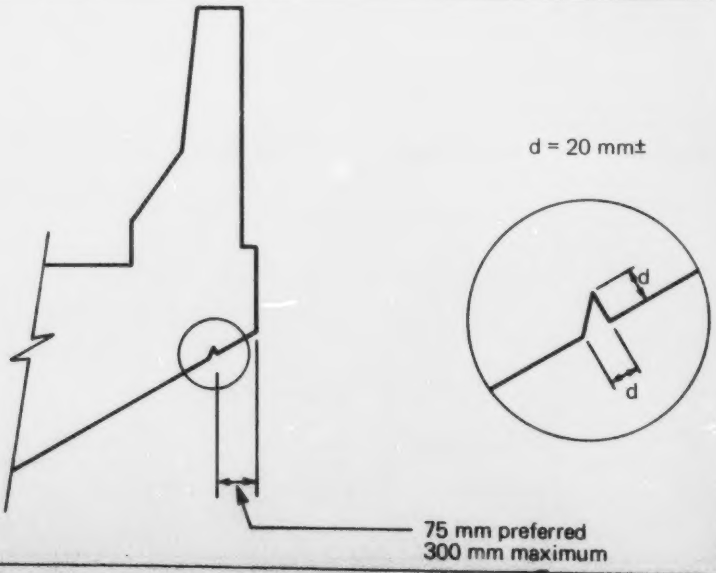
Downpipes should be avoided, wherever possible, in order to minimize the maintenance required to keep them clear of sand and debris washed from the road surface, and to avoid problems caused by freezing of water in the pipe.

Defective downpipe systems have caused maintenance problems, and their design merits more attention than it has received in the past. Chapter E of the MTC *Drainage Manual* [4] summarizes desirable design practice.

C1 — 4.7.5 Discharge From Deck Drains

Surface water from improperly located deck drains can cause traffic hazards, inconvenience to pedestrians and cyclists, erosion, and salt damage. Furthermore, runoff from the bridge deck and approach road can cause serious erosion and undermining of substructures, and should be accommodated by a properly designed drainage system.

Figure **Suggested Drip Groove Detail** **C1 — 4.7.6**



C1 — 4.7.6 **Drip Grooves**

Properly detailed and located drip grooves to prevent salt-laden water from flowing onto the underside of concrete bridge decks can reduce damage to the concrete. Suggested details are shown in Figure C1 — 4.7.6.

Maintenance and Inspection Requirements **C1 — 4.8**

C1 — 4.8.1 **General**

To facilitate inspection and maintenance, all temporary formwork in the vicinity of bearings, such as polystyrene, should be removed.

C1 — 4.8.2 **Deck Details at Joints**

Suggested deck details at joints are shown in Figures C1 — 4.8.2(a) to (d).

C1 — 4.8.3 **Bridge Bearing Seats**

The purpose of sloping the concrete surfaces between bridge bearings is to allow any water which passes through the deck joints to drain away from the bearings.

C1 — 4.8.4 **Provision for Jacking**

Sufficient space should be provided for placement of jacks at suitable jacking points. Steel or concrete diaphragms should be proportioned to resist the jacking forces or should be detailed to permit future strengthening.

C1 — 4.8.5 **Access to Box Girders**

Suggested details for access openings in steel and concrete box sections are shown in Figures C1 — 4.8.5(a) and (b).

Deck-Supported Utilities **C1 — 4.9**

Sewers and water lines should not be allowed inside bridge members such as box girders on account of the possibility of breakage. Gas and oil lines are prohibited on account of fire and explosion hazards.

Approach Slabs **C1 — 4.10**

Approach slabs 6 m long have been found to be adequate for reducing bumps at bridge ends caused by settlement of approach fills.

Concrete Bridge Deck Protection **C1 — 4.11**

Methods of protecting bridge decks against spalling are described in Reference 6. Provision of adequate cover to the steel is essential.

Figure **Deck Detail at Abutment** **C1 — 4.8.2(a)**

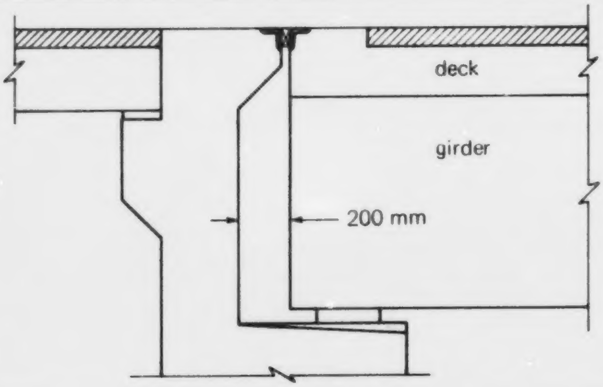
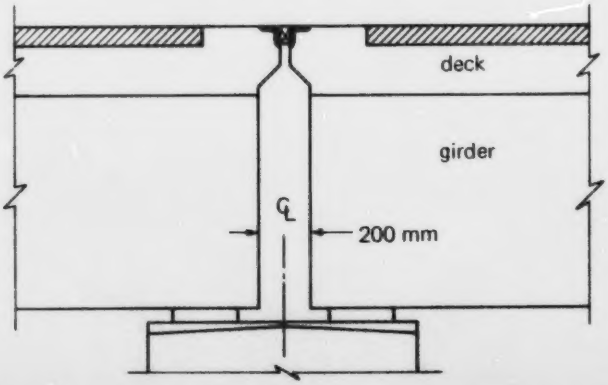


Figure **Deck Detail at Pier** **C1 — 4.8.2(b)**



Bridge Clearances — C1 — 5

Horizontal Clearances for Roadways Under Bridges — C1 — 5.1

C1 — 5.1.1 **Freeways**

C1 — 5.1.1.1 **General**

The horizontal clearance requirement of 10.0 m to abutments or piers is slightly greater than that in AASHTO [7] and is consistent with the Ministry's policy. The minimum clearance to toe of embankment slope, shoulder width plus 1.5 m, is also consistent with the [illegible]'s policy. This infringes on AASHTO clearance requirements, but has been [illegible] satisfactory. The clearance is to facilitate addition of a future traffic lane but m[illegible]educed, as indicated by Note 3 of Figure 1 — 5.1.2, if the median can accommodate the future lane.

C1 — 5.1.1.2 **Clearance to Median Pier**

These requirements are in accordance with the Ministry's policy and provide the maximum possible clearance to a pier in a narrow median. When the distance from edge of pavement to the pier is 10.0 m or greater, it is felt that an out-of-control vehicle will have slowed down significantly or regained control before hitting the pier.

C1 — 5.1.1.3

Adherence to the requirements of Figure 1 — 5.1.1 in the case of large skews can result in uneconomically long bridges. In such cases, this clause permits reduced clearances if guiderail is used.

C1 — 5.1.2 **Roads Other Than Freeways**

C1 — 5.1.2.1 **General**

This clause reflects the Ministry's policy on minimum acceptable clearances with guiderail protection. The minimum clearance of 0.5 m from the front face of flexible guiderail to the face of abutment or pier permits deflection of the guiderail on impact.

Figure **Deck Detail at Suspended Span — Girder Seat** **C1 — 4.8.2(c)**

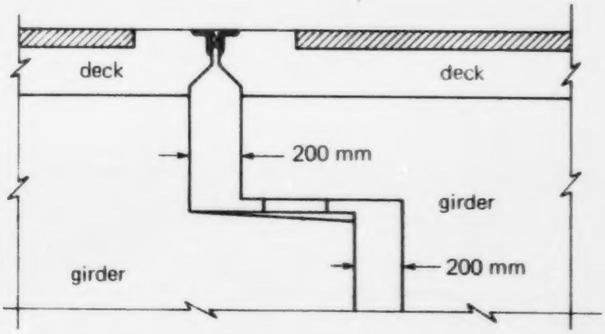
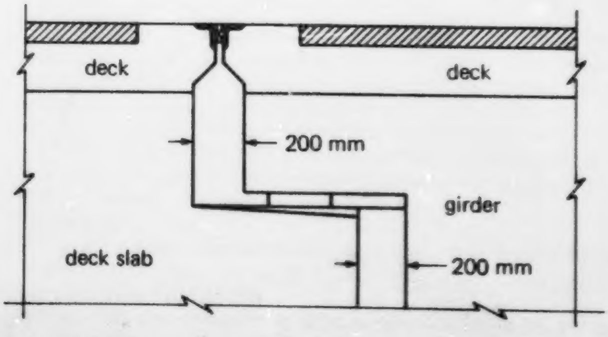


Figure **Deck Detail at Suspended Span — Slab Seat** **C1 — 4.8.2(d)**



For convenience, in Figure 1 — 5.1.2, clearances to flexible guiderail are shown on the right side of the diagram and to concrete safety profile barrier wall on the left side. The clearances shown apply to either left or right shoulders.

C1 — 5.1.2.2 **Sidewalks and Bikeways**

The reasons for the provisions of (a), (b) and (c) of this clause are the same as those for Clause 1 — 4.1.3. The minimum clearance of 1.0 m in (d) is in accordance with the Ministry's policy, which also stipulates a desirable clearance of 1.5 m. Sidewalks are usually raised under railway bridges as described in (d).

Vertical Clearance for Bridges **C1 — 5.2**

C1 — 5.2.1 **Over Roadways**

The vertical clearance for the more vulnerable bridges over roadways has been increased to 5.0 m from the previous standard clearance of 4.65 m. The increased clearance is in line with the present AASHTO [7] requirement of 4.88 m for major highways. It is expected that the increased clearance will reduce the number of incidents of damage to bridges by tall loads.

Figure **Access Opening for Precast Trapezoidal Girder** **C1 — 4.8.5(a)**

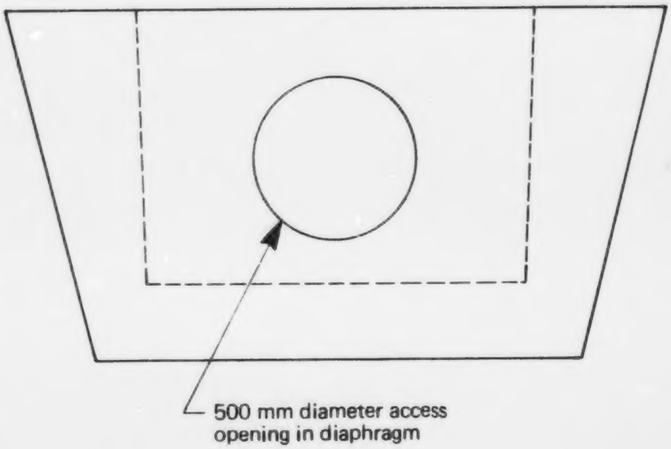
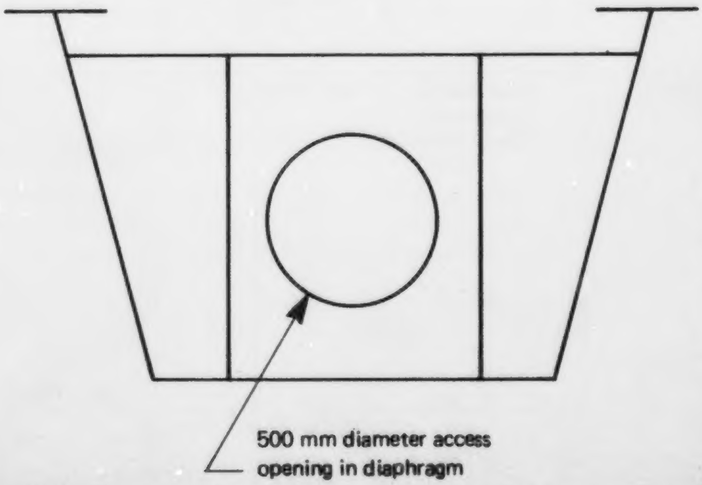


Figure **Access Opening for Steel Box Girder** **C1 — 4.8.5(b)**



C1 — 5.2.2 **Over Sidewalks and Bikeways**

A clearance of 3.0 m is considered desirable, but may be uneconomical in some situations. In most cases, the roadway clearance of 5.0 m will govern the bridge profile, and the provision of 3.0 m clearance over sidewalks and bikeways will not be a problem.

Vertical Clearance under Pedestrian and Bicycle Bridges **C1 — 5.3**

The clearance under pedestrian and bicycle bridges is greater than for vehicular bridges because the former are generally lighter than vehicular bridges and are therefore likely to sustain greater damage if hit by high vehicles.

Clearances for Bridges Over Water **C1 — 5.5**

C1 — 5.5.2 **Navigational Clearance**

Clearances for navigable waterways should conform to the requirements of the Navigable Waters Protection Act administered by Transport Canada. Requirements of local agencies such as conservation authorities or canal authorities should also be considered.

C1 — 5.5.3 **Clearances for Pathways under Bridges Crossing Water**

The minimum width of 1.5 m for pedestrian and bicycle paths under bridges crossing water is based on a single bicycle lane 1.0 m wide plus a clearance of 0.25 m from a curb, railing, or wall on each side. This basically agrees with the AASHTO recommendation [3]. A width of 1.0 m would be sufficient for pedestrians alone, but in most cases both pedestrians and cyclists have to be served, therefore the larger value was selected.

The minimum vertical clearance of 2.5 m is also in accordance with AASHTO [3]; 3.0 m is the desirable clearance.

Clearances for Bridges Over Railways **C1 — 5.6**

Railway clearances have been standardized and in the majority of cases the following clearances are required.

- Horizontal: 4.27 m centre-to-centre of tracks and 3.35 m to piers or abutments; 2.59 m during construction.
- Vertical: 7.16 m to base of rail; 6.58 m during construction.

Construction Clearances for Bridges Over Roadways **C1 — 5.7**

Clearances given in this clause have been the Ministry's standards for many years, and have been found satisfactory when used in combination with lower speed limits at construction sites.

Bridge Railings **C1 — 6**

General Requirements **C1 — 6.1**

The principal functions of bridge railings range from restraining vehicles and pedestrians to delineating the edge of the bridge deck on minor roads. This clause gives the owner considerable flexibility in choosing the most appropriate type of railing.

Railings which do not meet the full requirements of the Code may be considered for single lane bridges, where the impact angle is reduced, and on low traffic-volume roads in remote areas. A single line of flex-beam guiderail attached to steel or wood posts or an all wood system would be acceptable.

Combination Railings **C1 — 6.2**

Figure 1 — 6.2 shows combination railings most often used by the Ministry. Of these, the safety profile barrier wall types are used on virtually all provincial highway bridges, including those with sidewalks. Other types may be used by municipalities.

While the primary purpose of combination railings is to contain vehicles and pedestrians using the structure, consideration should be given to the protection of occupants of a vehicle in collision with the railing, other vehicles near the collision, and vehicles or pedestrians on roadways passing under the bridge.

When traffic using an intersecting roadway enters at or near the end of a bridge, railings meeting the requirements of this clause may restrict the visibility of such traffic; the location and geometry of intersecting roadways must be such that adequate visibility is provided.

C1 — 6.2.1 **Height of Barrier Walls**

Barrier wall heights specified meet current standards of the Ministry and AASHTO [7].

C1 — 6.2.2 **Geometry of Railing Elements**

a) Traffic rails are restricted to a height range of 375 mm to 850 mm for the purpose of distributing the transverse highway loading. The lower limit is similar to that given in AASHTO [7].

The upper limit ensures that sufficient rail resistance is provided to retain the load in the height range in which the impacting forces are applied to the railing. This dimension is based on the height of traffic rail in railings of this type currently permitted by the Ministry. It permits the location of a pedestrian rail at a practical spacing above the traffic rails to meet the overall height requirements of this clause. The height range restriction for the lowest traffic rail ensures that the bumpers of most passenger cars will strike it on impact.

The minimum overall height requirement for combination railings is greater than that of CSA S6 [8] but is similar to that of AASHTO [7] and the Ministry.

b) The restriction on the horizontal location of rails is identical to the requirements of AASHTO [7] and is necessary to ensure that all traffic rails will contribute to the resistance of the railing on impact by a vehicle.

c) The maximum clearance of 150 mm between rails and between the lower rail and the sidewalk surface is more restrictive than that in AASHTO but agrees with that in CSA S6 [8], and is considered essential for the safety of small children; it is also consistent with the size of rail members normally used.

C1 — 6.2.3 **Urban Freeway Bridges**

Where pedestrian walkways or bikeways are incorporated into urban freeway bridges, a rigid non-deflecting type of railing should be provided to protect users from errant vehicles. A 1.0 m high barrier wall with sloping safety profile provides this protection.

C1 — 6.2.4 **Transitions to Railings**

Avoidance of sharp changes in railing geometry or rigidity reduces hazards to vehicles and passengers.

Pedestrian Railings **C1 — 6.3**

C1 — 6.3.1 **General**

Pedestrian railings should be specified where they are not subject to impact from vehicular traffic. On railway bridges over roadways, the railings do not have to meet the requirements of Clauses 1 — 6.3.2 and 1 — 6.3.3.

C1 — 6.3.2 **Height of Pedestrian Railings**

The minimum overall height requirement is similar to that of ASSHTO [7] and is greater than that of CSA S6 [8].

C1 — 6.3.3 **Clear Opening in Railings**

The maximum opening requirement of 150 mm is more restrictive than that of AASHTO [7] but agrees with that of CSA S6 [8], and is considered essential for the safety of small children.

Hydraulic Design **C1 — 7**

Field Investigations **C1 — 7.1**

Detailed guidance on field investigations for bridges and large culverts may be found in *Field Hydrology Investigations for River Crossings* [9]. The same principles generally apply to smaller culverts, although the investigations for these should normally be less comprehensive.

Design Floods **C1 — 7.2**

C1 — 7.2.1 **Design Flood Discharge**

The method used for calculating design flood discharges depends largely on the size of basin, magnitude of flood involved, and whether the basin is rural or urban.

One problem facing designers is the choice of method out of the hundreds available, most of which are less than completely satisfactory. Bearing in mind the needs of a wide range of potential users, Table C1 — 7.2.1 summarizes methods considered to be suitable for the full range of runoff conditions in Ontario applicable to bridges and culverts. Omission of any particular method does not necessarily imply that it is inferior to those listed.

Chapter C of the MTC *Bridge Hydraulics Manual* [10] presents methods of design flood estimation for medium and large watersheds, while Chapter B of the MTC *Drainage Manual* [4] covers small watersheds.

C1 — 7.2.2 **Design High Water Level**

C1 — 7.2.2.1 **For Hydraulic Computations**

Neglect of the provision in Clause 1 — 7.2.2.1(a) can, at some crossings, lead to structure failure due to excessive scour.

C1 — 7.2.2.2 **For Establishing Soffit and Approach Grade**

Occasionally, the high water level caused by ice jams may be considerably higher than that under ice-free conditions, in which case the soffit and approach grade should be raised accordingly.

Table **Computation Methods for Design Flood Discharges for Bridges and Culverts** **C1 — 7.2.1**

Method	Comments
Station Frequency Analysis	Preferred method where suitable flow records are available, for any size of basin, and return periods up to 100 years [10]
Watershed Classification Method	Based on regional frequency analysis. Preferred method where station frequency analysis cannot be used, for basins over 25 km² and return periods up to 100 years [10].
Unit Hydrographs	For return periods exceeding 100 years, particularly where regional design floods must be considered. Hydrographs based on recorded data are preferred to synthetic hydrographs whenever circumstances permit their use [4, 10].
Rational Method	For basins of area up to 25 km² where the other methods listed are not feasible. Can give acceptable results in many cases if properly used [4].
Miscellaneous Methods	For checking design flood estimates made by other methods, and for calculating runoff in special cases where normal hydrologic methods are not applicable [10].

If backwater is not considered in establishing the road grade, the freeboard on the upstream side will be less than that on the downstream side.

Hydraulic Design of Bridges C1 — 7.3

C1 — 7.3.1 General

Consideration of scour and related phenomena is essential at most stream crossings, whereas detailed consideration of backwater, which can be time-consuming, is necessary only at relatively few crossings where upstream flooding may be critical.

C1 — 7.3.2 Design for Scour and Related Factors

Table **Competent Mean Velocities for Cohesive Soils*** **C1 — 7.3.2**

	Soil Scourability**		
Depth of Flow	High	Medium	Low
m	m/s	m/s	m/s
1.0	0.5	0.9	1.6
1.5	0.6	1.0	1.8
3.0	0.6	1.2	2.0
6.0	0.7	1.3	2.3
15.0	0.8	1.5	2.6

* Competent velocities should be based on local experience whenever possible, taking into account saturation and weathering.

** It is not considered advisable to relate the tabulated values to soil property indices because of the strong effect of saturation and weathering on the scourability of soils. However, the following tentative relationship to soil consistency is offered as a rough guide.

- High scourability — very soft to soft clays.
- Medium scourability — firm to stiff clays.
- Low scourability — stiff to hard clays; some glacial tills.

The above information is based on the RTAC *Guide to Bridge Hydraulics*[1]. Soil consistency can be judged by the following field test applied with the soil at or near its natural water content [11].

- Very soft — easily penetrated several centimetres by fist.
- Soft — easily penetrated several centimetres by thumb.
- Firm — moderate effort required to penetrate several centimetres.
- Stiff — readily indented but penetrated only by great effort.
- Very Stiff — readily indented by thumbnail.
- Hard — indented with difficulty by thumbnail.

C1 — 7.3.2.1 **General**

a) Failure to take account of scour and related factors is a common cause of washouts of bridges and open-footing culverts.

General scour, caused by constriction of the flow, is the most common type of scour at stream crossings. Local scour, caused by the presence of piers, must be added to the general scour as illustrated by Figures C1 — 7.3.2(i) and (ii). If the structure waterway is too large to create general scour, the possibility of natural scour, as discussed in Clause C1 — 7.3.2.5, should be considered. Degradation and artificial deepening have caused several structure failures in the past in relatively limited areas of the province.

b) This clause is necessary to ensure that the scour is not measured below that already created by an existing structure.

c) Local scour at a pier takes place below the level of general or natural scour at that point and, therefore, must be added to it. If there is no pier, the total scour depth required for the design of the abutment scour protection is the greater of the maximum general scour and the maximum natural scour (and, in most cases, will be the former).

To simplify the computations, the effects of streambed degradation and artificial deepening are accounted for in the design of the scour protection (Clause 1 — 7.3.3).

C1 — 7.3.2.2 **High Water Level for Scour Calculations**

Use of the incorrect high water level at a crossing which experiences abnormal flood stage conditions can result in excessive scour and failure of the structure.

Figure **General and Local Scour at a Bridge** **C1 — 7.3.2(i)**

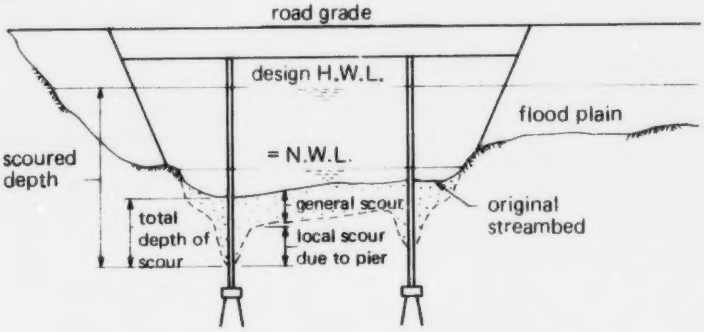
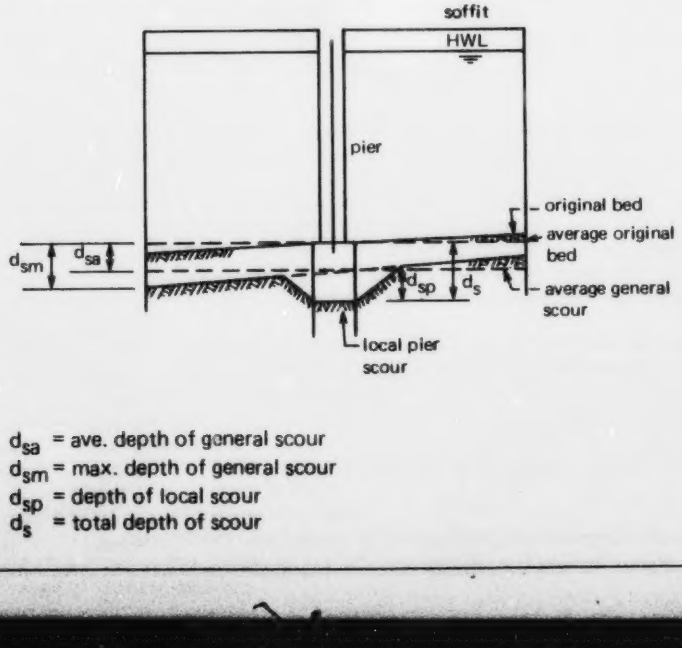

road grade
design H.W.L.
flood plain
scoured depth
= N.W.L.
original streambed
total depth of scour
general scour
local scour due to pier

Figure **Typical Scour at a Bridge with Spread Footings** **C1 — 7.3.2(ii)**


soffit
HWL
pier
original bed
average original bed
average general scour
local pier scour
d_{sm}
d_{sa}
d_{sp}
d_s

d_{sa} = ave. depth of general scour
d_{sm} = max. depth of general scour
d_{sp} = depth of local scour
d_s = total depth of scour

C1 — 7.3.2.4 **General Scour**

a) The competent velocity method given in the *Guide to Bridge Hydraulics* [1] has been found suitable for calculating the average depth of general scour at most stream crossings in Ontario. However, there is a possibility that the competent velocities for cohesionless soils given by revised Figure 4 — 12 of the Guide [1] may be too high, and could result in under-estimation of scour depths.

For this reason, both the curves of competent velocity in Reference 1 and those used by the Ministry are given in Figure C1 — 7.3.2(iii). The selection of a suitable competent velocity at a given crossing should take into consideration experience at other structures in the vicinity having similar hydraulic conditions. Table C1 — 7.3.2 for cohesive soils is largely based on the Guide [1], and is found to give acceptable results in practice.

b) Maximum general scour at a structure site may be estimated by redistributing the average general scour in accordance with Figure 4 — 13 and page 93 of the Guide [1], but the triangular redistribution should be used only at crossings located on sharp bends of the channel. In most cases, the maximum scour will be less than that given by the triangular shape and the distribution has to be decided on the basis of experience and judgement.

c) During the life of a bridge a meandering channel passing through highly erodible alluvial soil may change its course drastically, with the result that the deepest scour changes from one side of a bridge to the other. Scour protection should account for this possibility.

d) Blockage of a channel by an ice jam can force the flow to pass under the ice, thereby creating general scour if the bed is erodible. If this scour is likely to exceed the general scour resulting from ice-free conditions, the larger value should be used for determining the ultimate bed elevation.

C1 — 7.3.2.5 **Natural Scour**

a) Even if a bridge waterway were large enough not to constrict the flow and therefore not to cause general scour, the natural erosion processes of the river would still create natural scour which could endanger the structure. This situation rarely arises in Ontario because some degree of constriction is nearly always present but, if required, the depth of natural scour can be estimated according to methods in the *Guide to Bridge Hydraulics* [1].

b) Channel shift can occur as discussed in Clause C1 — 7.3.2.4(c).

C1 — 7.3.2.6 **Local Scour at Piers**

The formula and Tables 1 — 7.3.2(a) and (b) in the Code are based on information in the *Guide to Bridge Hydraulics* [1].

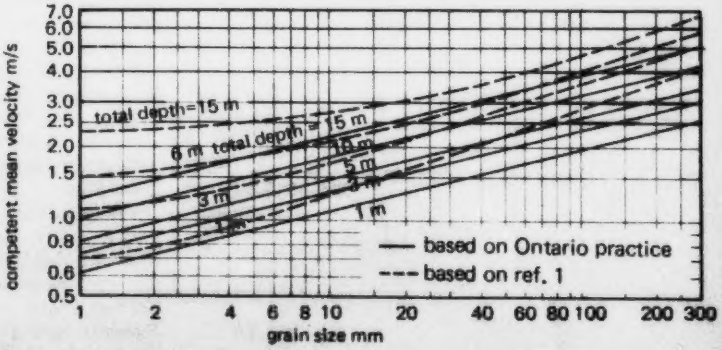
The state of knowledge of local scour at abutments and in cohesive materials is not sufficiently advanced to permit recommending specific methods. Inspection and measurement of local scour at existing structures on the same stream should give the best guidance as to depths to be expected at a new structure.

Local scour is illustrated in Figures C1 — 7.3.2(i) and (ii).

C1 — 7.3.2.7 **Degradation**

Degradation, or general lowering of a streambed over a long period, is a serious problem which should be considered in the design of structures on degrading channels. The amount of degradation may exceed 10 m over a period of several dec-

Figure **Competent Mean Velocities for Cohesionless Soils** **C1 — 7.3.2(iii)**



ades. There appears to be no reliable method of calculating degradation analytically at the present, and reliance should be placed on field observations. Reference 12 summarizes the current state of the art.

C1 – 7.3.2.8 Artificial Deepening

Artificial deepening of municipal drains and other channels has, in the past, caused undermining and collapse of structures, and every effort should be made to anticipate such deepening in the design of new structures. This is also necessary in areas where urban drainage schemes may result in deepening of several metres. If drainage plans are not available, reliance should be placed on field observations, discussions with local authorities, and past experience. Allowance for major future deepening should be made only where there are definite plans for the deepening.

C1 – 7.3.3 Protection Against Scour

C1 – 7.3.3.1 General

Protection against undermining of structure foundations by scour may be achieved by providing sufficient depth of spread footings, or by the use of piles, sheet piles, or paved inverts.

C1 – 7.3.3.2 Spread Footings

Spread footings in erodible soils are vulnerable to floods larger than the design flood, but their lower initial cost usually favours their use where the soil bearing capacity is suitable, except on highly scourable soils such as sand, silt, or fine gravel.

1/ Depth of Footings

a) i) The depths given correspond to current practice or are slightly greater as a result of conversion to the metric system. Specific mention has been made of concrete wingwalls because these sometimes fail due to an inadequate depth of footing. Footing depths should be measured below the original level of the streambed to avoid the excessive footing depths which would result if they were measured below the scoured bed. Allowance for bed degradation and artificial deepening is made in (d).

ii) The factor of 1.7 allows a theoretical safety margin of at least 0.4 times the depth of footing before the footing starts to become undermined. It is intended to allow for faulty design assumptions, inadequacies in the scour computation methods, and the probability that the structure will experience a flood larger than the design flood during its lifetime.

c) Bridges with gabion or timber crib substructures are usually shorter-lived lower-cost structures, and it is difficult to justify placing the foundations at the full depth required for more costly structures. The 50% reduction in footing depth is an arbitrary one based on experience.

2/ Limitations on Use of Spread Footings

a) Structures with spread footings on highly erodible soils are very vulnerable to failure due to scour and undermining, and should not be used without the precautions specified.

b) Bridges founded on erodible material at a level higher than the streambed are vulnerable to failure if the streambanks become eroded as is often the case. The bank protection required for such designs must remain effective for the lifetime of the bridge.

C1 – 7.3.3.3 Piles

Although piles do not provide a complete guarantee against scour failure, they can withstand much greater depths of scour than spread footings. The increased resistance to scour failure of bridges on piles may also permit a reduction of span below that required for spread footings. Piled foundations also have the advantage that the footings can often be placed at a shallower depth, thereby facilitating dewatering.

Although piled abutments are relatively safe even if undermined, the fill behind the abutments should be prevented from washing out and causing a hazardous cavity in or below the road surface.

C1 – 7.3.3.4 Sheet Piles

The provisions are intended to prevent failure due to scouring of material in front of the sheet piling. If sheet piles are required in the dewatering of excavations, it is often desirable to leave them in place permanently as scour protection. Their provision solely for scour protection is rarely justified except to prevent undermining of piled abutments.

C1 – 7.3.3.5 Protective Aprons

The provisions in this clause are based on the *Guide to Bridge Hydraulics* [1]. Procedures for determining the size of stone are given in Appendix IV of the Guide.

C1 — 7.3.4 Design for Backwater

C1 — 7.3.4.1 General

The recommended method for manual calculation of backwater at bridges is given in Reference 13; the computer method is outlined in Program HY 720, *Hydraulics of Bridge Waterways* [14], or equivalent programs. Channel backwater calculations, where required, should be performed manually by methods given in standard texts such as *Open-Channel Hydraulics* [15], or computed by programs such as HEC 2, *Water Surface Profile* [16].

C1 — 7.3.4.3 Depth of Scour Used in Backwater Calculations

The increase of bridge waterway area created by scouring of the streambed has the beneficial effect of reducing the amount of backwater. However, as scour is a function of time, the calculated depth of scour may not develop during a single design flood; for this reason, the Code specifies that normally only half the calculated average depth of general scour should be used in the backwater calculations.

C1 — 7.3.4.4 Enlargement of Waterway

Artificial enlargement of the bridge waterway opening by widening or deepening the channel is beneficial in reducing backwater, but care should be taken not to create erosion or deposition problems in the upstream or downstream channel.

C1 — 7.3.4.5 Reduction of Backwater by Relief Flow

Relief flow over the approach road to a structure can greatly reduce backwater during a regional flood and should, therefore, be maximized by keeping the grade as low as possible, provided that the minimum freeboard requirements for the normal design flood are satisfied. The comparison between crossings with and without relief flow is illustrated by Figures C1 — 7.3.5(i) and C1 — 7.3.5(ii).

C1 — 7.3.5 Other Hydraulic Requirements for Bridges

C1 — 7.3.5.1 Location and Alignment

b) Whenever possible, piers and abutments should be located and aligned so as not to affect adjacent structures adversely, or be affected by them. This is particularly important if the structure is immediately downstream from a dam spillway.

c) The location and alignment of piers and abutments may affect scour depths considerably, particularly if soils are of variable scourability across the proposed bridge opening.

d) Costly claims of erosion damage to downstream property can arise from improper location and alignment of a structure.

Figure **Clearance and Freeboard for Regional Design Flood with Maximum Relief Flow** **C1 — 7.3.5(i)**

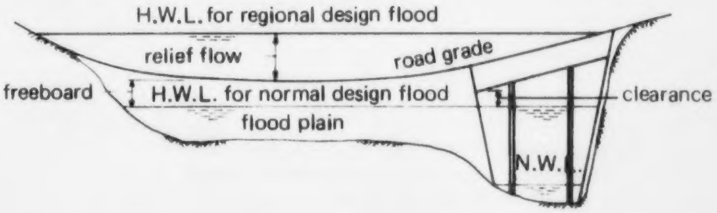
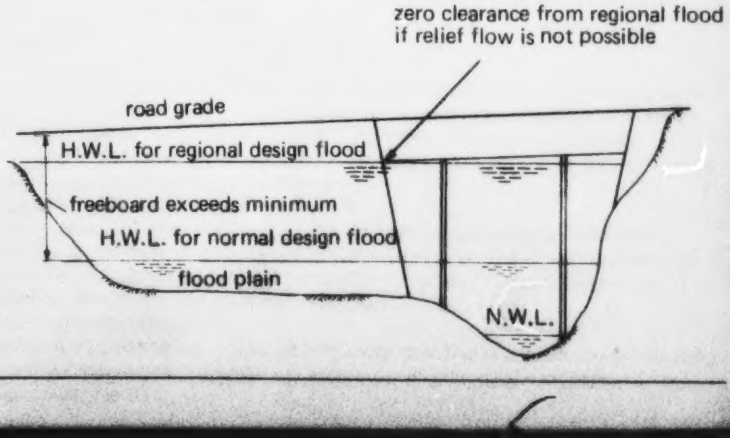

H.W.L. for regional design flood
relief flow
road grade
freeboard
H.W.L. for normal design flood
clearance
flood plain
N.W.L.

Figure **Clearance and Freeboard for Regional Design Flood with no Relief Flow Possible** **C1 — 7.3.5(ii)**


zero clearance from regional flood
if relief flow is not possible
road grade
H.W.L. for regional design flood
freeboard exceeds minimum
H.W.L. for normal design flood
flood plain
N.W.L.

C1 — 7.3.5.2 **Vertical Clearance**

a) The clearances specified are in accordance with the Ministry's current practice.

b) This situation is illustrated in Figures C1 — 7.3.5(i) and C1 — 7.3.5(ii). If sufficient relief flow is possible during a regional flood, the structure cost can be greatly reduced by designing it to accommodate only the normal design flood. However, if the vertical alignment of the road is such that relief flow is not possible, the structure should be high enough and large enough to accommodate the regional flood. In this case, the clearances specified in (a) above are not necessary because the regional flood is usually much larger than the normal design flood. In other cases where only a moderate amount of relief flow is possible, the structure opening may have to be designed for a discharge between the normal and the regional design flood.

c) This provision ensures that sufficient clearance is available at low flows to permit proper inspection of the underside of the structure.

d) Specifying clearances for arch structures is not practicable due to the wide range of circumstances. For example, in the case of open steel arches, it is essential to prevent a build-up of water and ice against the superstructure; whereas, in the case of earth-filled arches it is often feasible to have much of the soffit submerged. Each arch crossing should therefore be treated on its own merits.

C1 — 7.3.5.3 **Piers and Abutments**

Information on bridge protection devices can be found in Reference 17.

C1 — 7.3.5.4 **Buoyancy and Lateral Displacement**

Timber bridge superstructures, and concrete and steel superstructures which entrap air when submerged by floodwaters, are susceptible to being floated and carried downstream. The precautions specified in this clause are intended to minimize this possibility, and are similar to those recommended in *Hydraulics of Bridge Waterways* [13].

Hydraulic Design of Culverts

C1 — 7.4

In this section, culverts have been considered under two main classes: closed-invert culverts, and open-footing culverts with scourable inverts. Open-footing culverts with non-scourable inverts should be designed as closed-invert culverts. Experience has shown that open-footing culverts with scourable inverts are vulnerable to failure resulting from undermining of the footings by scour, degradation, and artificial deepening. On the other hand, properly designed and installed closed-invert culverts are virtually indestructible and should survive floods much greater than the design flood unless the road itself washes out. For this reason, closed-invert culverts should be used wherever possible.

C1 — 7.4.1 **Closed-Invert Culverts**

C1 — 7.4.1.1 **General**

Chapter D of the MTC *Drainage Manual* [4] covers all normal aspects of the hydraulic design of culverts, including manual and computer procedures for conventional and improved inlet culverts, fish passage considerations, relief flow computations, and the design of end treatment.

The full potential of improved culvert inlets for reducing culvert costs has not been realized in the past. Chapter D attempts to remedy this by presenting a design procedure which encourages the designer to consider improved inlets wherever site conditions favour their use. In some cases, such inlets can increase culvert capacities by 100%, or barrel sizes may be reduced accordingly. However, the reduced barrel cost must be weighed against the increase cost of the inlet and outlet structures.

Properly bevelled edges should be provided on inlets of all *in situ* concrete box culverts, whether conventional or with improved inlets.

It should be noted that the computation of backwater at culverts using the U.S. Corps of Engineers computer program HEC 2, *Water Surface Profile* [16], as done on many floodline mapping studies in Ontario, is reported to give widely different results from those given by the methods in Chapter D. Until this problem can be resolved, it is recommended that more weight be given to the Chapter D solution [4], as this follows a more comprehensive procedure.

C1 — 7.4.1.2 **Allowable Backwater**

In cases where the flooding of buildings or developable land is not a factor, the allowable amount of backwater is often expressed as a proportion of the culvert height. Current practice in the Ministry is normally to permit conventional closed-invert culverts to flow full at the inlet. Consideration has been given to permitting such culverts to flow submerged during design floods, but this is generally considered undesirable due to the difficulty of removing debris under flood conditions, the reduced safety factor, and the possibility of piping failure. However, submergence of an existing culvert or its extension may obviate the need for an expensive replacement structure. Submergence may also be permissible at crossings where, for example, a large river downstream produces a water level at the site several metres higher than that created by the culvert.

In the case of improved inlet culverts, the allowable backwater is generally determined by consideration of potential flooding of up-stream property and, for maximum economy, the inlet may be submerged.

C1 — 7.4.2 End Treatment for Closed-Invert Culverts

Failures of closed-invert culverts usually occur as a result of uplift or undermining of culvert ends.

C1 — 7.4.2.1 Application

a) **Concrete Box Culverts**
   Cutoffs at the ends of concrete box culverts are essential in all cases to prevent undermining of the floor slab.

b) **Metal Culverts**
   The flexibility and light weight of corrugated metal culverts make them particularly vulnerable to failure due to hydraulic uplift of the inlet or upward buckling of the invert. However, special end treatment should be provided only when there is ample justification.
   i) Concrete cutoffs and partial headwalls have proved to be very successful, and are the preferred means of preventing uplift of metal culverts. However, at crossings where concrete may not be readily available, or where the placing of concrete may not be practicable, other end finishes are acceptable.
   ii) The simplest means of reducing the possibility of uplift is grading the fill so as to minimize the uncovered length of pipe. This is particularly necessary in the case of heavily skewed culverts, where the exposed length sometimes exceeds 15 m. Culvert ends projecting from the fill to accommodate future widening should also be covered by fill, unless other precautions against uplift are taken.
   iii) The provisions in this clause are necessary to prevent upward buckling of the invert due to hydraulic pressure.

C1 — 7.4.2.2 Details of End Treatment

a) **Concrete Cutoffs and Headwalls**
   Minimum depths specified in this clause are based on the Ministry's current practice.

c) **Attachment to Culvert**
   Lack of attachment of the culvert to the anchorage has been a factor in uplift failures.

d) **Special Requirements for Degrading Streams**
   Special outlet treatment for culverts on degrading streams is essential to prevent undermining and eventual failure.

C1 — 7.4.3 Open-Footing Culverts

C1 — 7.4.3.1 Hydraulic Design

Scour-related factors have rarely been adequately considered by designers in the past, giving rise to frequent problems on municipal drains and washouts on natural streams. Design procedures for open-footing culverts are provided in Chapter D of the MTC *Drainage Manual* [4].

C1 — 7.4.3.2 Vertical Clearance of Open-Footing Culverts

The clearance is beneficial in providing additional waterway area in a large flood, thereby reducing the risk of a scour failure. Some clearance is also desirable for smaller open-footing culverts.

C1 — 7.4.4 Other Requirements for Culverts

C1 — 7.4.4.1 Culvert Extensions

Extension of an existing multi-span culvert by a single span upstream, or extension of a single-span culvert by multiple spans downstream, can create serious debris blockage problems inside the culvert during floods, and should be avoided.

Changes of cross-sectional shape may also snag debris, but this may sometimes be overcome by providing gradual transitions.

C1 — 7.4.4.2 Alignment of Non-Linear Culverts

The maximum changes of direction specified are those suggested in AASHTO *Guidelines for the Hydraulic Design of Culverts* [18]. The limitation is necessary to prevent an accumulation of debris inside the culvert and to prevent undue loss of head in culverts operating with high velocities.

C1 — 7.4.4.3 Subdrains for Metal Culverts

Subdrains are necessary in the circumstances specified to prevent upward buckling of metal inverts under hydrostatic pressure, particularly in the case of pipe arch structures.

Freeboard **C1 — 7.5**

C1 — 7.5.1 **Normal Design Flood**

The specified minimum freeboards of 1.0 m and 0.3 m correspond with the Ministry's current practice. A typical layout is illustrated in Figure C1 — 7.5.1. The approach grades are kept as low as practicable to permit relief flow during floods greatly exceeding the design flood.

C1 — 7.5.2 **Regional Flood**

This clause was included to ensure that, whenever possible, the cost of complying with regional flood requirements is kept to a minimum by keeping the embankment height low and allowing as much relief flow over the roadway as possible. Figure 1 — 7.3.5(i) illustrates such a situation while Figure 1 — 7.3.5(ii) illustrates a typical increase of bridge size necessitated by lack of relief during the regional flood.

C1 — 7.5.3 **Freeboard for Pathways Under Bridges Crossing Water**

The amount of freeboard between water level and the surface of a pathway under a bridge may influence the amount of maintenance required, accessibility during periods of high water and, in the case of low bridges, the height of the superstructure. The possibility of recurring maintenance costs caused by frequent flooding may justify use of a greater freeboard.

Erosion Control **C1 — 7.6**

C1 — 7.6.1 **Bank Protection**

While the control of erosion at bridges and culverts is an important segment of the overall design, such measures should be applied only where justified since the cumulative cost of indiscriminate and widely applied control measures would be appreciable.

The design of bank protection is covered in Chapter C of the MTC *Drainage Manual* [4].

C1 — 7.6.2 **Protection Against Degradation**

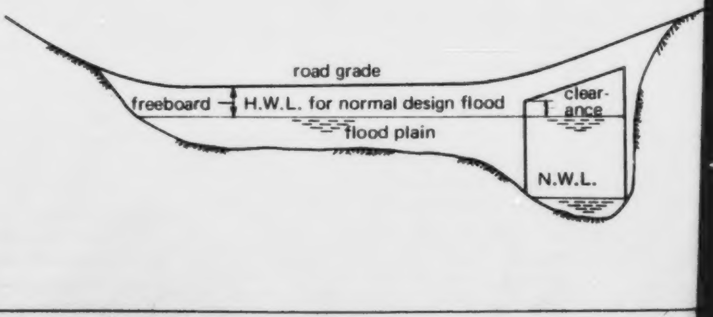
The problem of degradation was discussed in Clause C1 — 7.3.2.7. Properly designed highway culverts can provide a positive barrier to the upstream progression of gullies on actively degrading streams at a relatively low cost compared to that of alternative solutions such as constructing a series of check dams. Thus, the provisions of this clause help not only to protect the culvert against failure due to undermining, but also to prevent significant loss of what is often good agricultural land upstream of the highway.

The same provisions may be applied to bridges, although in this case the costs and benefits of the measures specified should be weighed against those of deepening spread footings and not controlling upstream progression of the degradation.

It is important that the downstream side of the structure be provided with a cutoff which is deeper than the anticipated depth of degradation. On very active gullies, the amount of deepening may be 10 m or more during the life of culvert. In this case, it is necessary to provide sufficient protection for at least 20 years, after which the culvert should be provided with a drop outlet structure or protected by other means.

Provision (d) is necessary to ensure that the new structure will not induce upstream degradation.

Figure **Freeboard and Clearance for Normal Design Flood** **C1 — 7.5.1**



C1 — 7.6.3 **Storm Sewer and Channel Outlets**

Storm sewers discharging inside open-footing structures may cause serious undermining if concrete pads or other suitable erosion control devices are not provided. Sewers and channels discharging immediately adjacent to structures may undermine the structure or the wingwalls.

Stream Diversions **C1 — 7.7**

Diversions may cause harmful erosion and sedimentation, as well as adverse environmental and hydrologic effects, and it is the Ministry's policy to construct them only if there is no other reasonable solution.

Requirements of Other Agencies **C1 — 7.8**

Evaluation of the requirements of other agencies is necessary to ensure that the cost to the Ministry is reasonable in relation to the benefit derived and that other, less costly alternatives are not feasible.

Estimates of discharges and high water levels provided by other agencies are sometimes questionable, depending on the assumptions made and the methods used.

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[2] ASCE, *Nomenclature for Hydraulics*, New York, N.Y., 1962.

[3] AASHTO, *Guide for Bicycle Routes*, Washington, D.C., 1974.

[4] Ontario Ministry of Transportation and Communications (MTC), *Drainage Manual*, Downsview, Ontario, 1982.

[5] MTC, *Structural Manual*, Downsview, Ontario.

[6] MTC, "Durable Bridge Decks: A Review of Progress in Ontario 1976-78," MTC, Downsview, Ontario, 1978.

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[18] ASSHTO, *Highway Drainage Guidelines*, Vol. IV, Hydraulic Design of Culverts, Washington, D.C., 1979.

C2 Loads

Clause	Contents
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C2.4	Loads
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]
C2.4	[illegible]

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Scope **C2 — 1**

With few exceptions, existing highway bridges in the Province of Ontario have individual spans less than 125 m. It is expected that this span will not be exceeded frequently in the future. Consequently, highway live loads for Section 2 have been developed and load factors calibrated for bridges in which no individual span exceeds 125 m.

Loads **C2 — 4**

Dead Loads **C2 — 4.2**

Dead load consists of the weight of all structural and nonstructural permanent parts of the bridge, including any additional parts expected in the future. For design of a highway bridge having an exposed concrete wearing surface, an added bituminous wearing surface should be considered even though there is no immediate intention to place such a layer. The weight of stay-in-place forms should not be overlooked.

C2 — 4.2.2

Unit material weights given in Section 2 are close to average values. In the case of structural steel sections embedded in concrete, the dead load of the member should be determined by using the unit weight of steel for the steel section and the unit weight of plain concrete for the concrete part of the member.

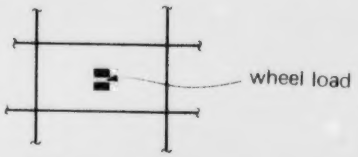
Live Load **C2 — 4.3**

C2 — 4.3.1 **Highway Live Loads**

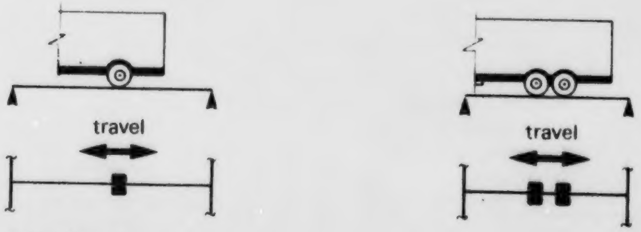
C2 — 4.3.1.1 **Highway Classification**

The criteria given in the first column of Table 2 — 4.3.1.1 have been derived specifically to meet the requirements of the Code. The criteria given in the second column are based upon those employed in *Geometic Design Standards for Canadian Roads and Streets* issued by the Roads and Transportation Association of Canada.

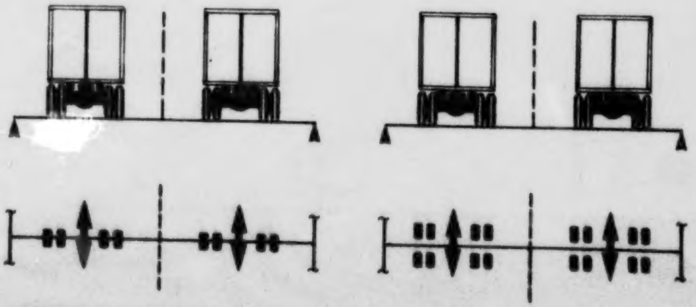
gure **Some Examples of Critical Loading for Deck Components** **C2 — 4.3.1.2(a)**



(a) deck slab



(b) stringers



(c) floor beams

C2—4.3.1.2

OHBD Load

Specified highway live loads should reflect actual vehicle loads and bear a direct relationship to the legal loads on the highway. Such relationships can be established and monitored through periodic surveys of vehicle weights and dimensions [1].

Highway traffic consists of a large number of light vehicles and a comparatively small number of heavy vehicles. Modern heavy vehicles include a wide variety of vehicle types ranging from short two-axle trucks to long truck trains. The gross weight of such trucks varies from about 120 kN to 650 kN. The usual overall length of a truck train is approximately 20 m.

On a single-lane bridge, critical loading for deck components and short-span structural components with span lengths up to 8 m is generally heavy wheel loads or axle loads in the design lane. For multi-lane bridges, the simultaneous presence of axle loads in more than one design lane may be more critical. Some examples of critical loading of deck components are shown in Figure C2—4.3.1.2(a).

Critical loading for main longitudinal components and support systems for intermediate span bridges, with span lengths varying from 8 m to 25 m, is generally a single heavy vehicle in a design lane. For long-span bridges, multiple presence of vehicles in a design lane may be more critical. Some examples of critical loading for the main longitudinal components are shown in Figure C2—4.3.1.2(b). For such components in a multi-lane bridge, simultaneous presence of vehicles in more than one design lane may be more critical, as shown in Figure C2—4.3.1.2(c).

Hence, specified highway live loads should model the following:

1/ heavy wheel loads;
2/ heavy axle loads in a design lane;
3/ one heavy vehicle in a design lane;
4/ multiple presence of vehicles in a design lane;
5/ simultaneous presence of vehicles or axle loads in more than one design lane.

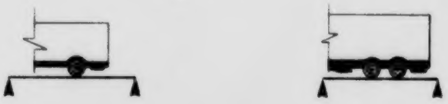
The highway live loads specified in the Code were selected from several possible loading models based on two primary characteristics; namely, simplicity in design applications and efficiency in modelling actual highway loads. The above considerations resulted in a highway live load system consisting of two alternative components, the Truck and the Lane Load.

The Truck models 1, 2 and 3, above, while 4 is modelled by the Lane Load. The presence of vehicles in more than one lane is accounted for by a modification factor described in Clause C2—4.3.1.9.

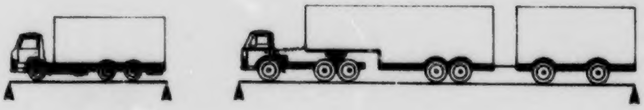
a) Truck

The idealized truck was developed to represent the many types and configurations of real trucks. There is an infinite number of possible combinations of axle weights

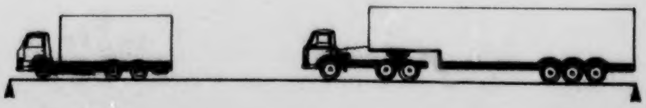
Figure **Some Examples of Critical Loading for Main Longitudinal Components** **C2—4.3.1.2(b)**



(a) short span components



(b) intermediate spans



(c) long spans

and spacings. The concept of 'Equivalent Base Length' permits transformation, which makes it easier to correlate the load effect of different trucks [1]. Equivalent base length B_m, illustrated in Figure C2—4.3.1.2(d), is defined as a length over which the total weight of a group of axles can be uniformly distributed to cause load effects in a bridge structure similar to those caused by the group of axles itself.

The legal vehicle weights in the Province of Ontario [2] are based upon the Ontario Bridge Formula (OBF):

$$W_m = 9.806\,(10.0 + 3.0\,B_m - 0.0325\,B_m)$$

where: W_m = permissible total weight on a group of axles, kN,
B_m = equivalent base length of the group axles, m.

From various load surveys in Ontario [1, 3], a relationship between the legal and the actual highway loads has been established. The level of maximum observed load (MOL) is found to be approximately 100 kN greater than indicated by the Ontario Bridge Formula and is given by the following formula:

$$W_{MOL} = 9.806\,(20.0 + 3.0\,B_m - 0.0325\,B_m)$$

The MOL level serves as the basis of the current special overload permits in Ontario.

The Ontario Highway Bridge Design Truck, referred to below as the OHBD Truck, is an upperbound representation of modern heavy trucks and is based upon the MOL level. Figure C2—4.3.1.2(e) displays the subconfigurations of the OHBD Truck along with the OBF and the MOL levels.

The most common axle units used in heavy trucks are single and dual axles. A dual axle consists of two consecutive axles articulated through a common suspension designed to equalize the weight on the axles. The axle units were established using statistical information on axle weights from various load surveys.

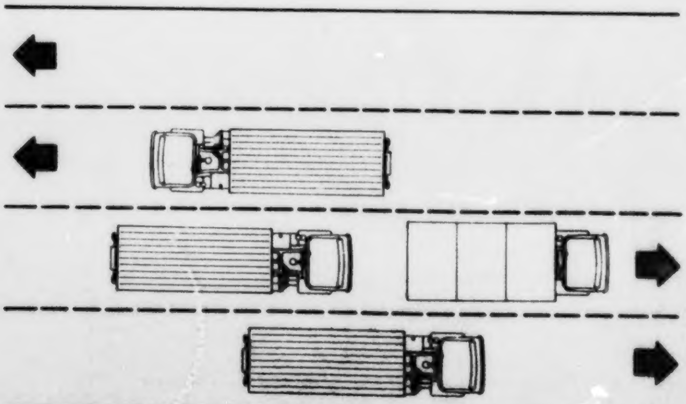
Triple axles are also used in modern vehicles in limited numbers, but their effect is adequately modelled by the dual axle unit in this Code.

Using the OHBD Truck it is possible to maintain a link between the Bridge Design Code and the Highway Traffic Act and Regulations for heavy vehicle weight control in Ontario.

b) Lane Load

For development of the lane load, it is essential to consider the load effect of the diverse traffic mix in a lane on bridges of various types and geometry. Because traffic loading on bridges is a random process, the largest effects that will occur can be predicted only in a statistical manner. In this approach, "mean largest effect" is an important statistical parameter. It is defined as an average of effects, each being the largest effect occurring in the same member of separate but identical bridges subjected to the same volume of traffic during a specified period.

Figure **Simultaneous Presence of Vehicles in More Than One Design Lane** **C2—4.3.1.2(c)**



Several methods have been used for predicting the mean largest effects for influence lines of various shapes and sizes. In Figures C2 — 4.3.1.2(f) and C2 — 4.3.1.2(g), mean largest effects for a chosen lifetime of 50 years, calculated by two different methods, are plotted against the maximum effect of a unit uniformly-distributed load; both scales are normalized by dividing the values by the corresponding maximum effect of one OHBD Truck.

The values plotted in Figure C2 — 4.3.1.2(f) were calculated using a probabilistic method [4] based on a study of 6825 vehicles from the 1975 survey.

The values plotted in Figure C2 — 4.3.1.2(g) were calculated using the eight heaviest trucks from the 1975 survey, each loaded above the MOL level [5]. Maximum effects were calculated by placing up to three trucks in a lane at a minimum clear distance of 10 m. The effects were statistically adjusted to account for the expected number of one, two and three truck events in a bridge lifetime.

In Figures C2 — 4.3.1.2(f) and C2 — 4.3.1.2(g), the effect of the Lane Load specified in the Code for Class A highways is also shown. The lane loads for Class B and C highways were developed in a similar manner using smaller traffic volumes.

Variability of the large effects as well as the ratio of the mean largest effect to the effect caused by the OHBD load is accounted for in the live load factor [6, 7].

C2 — 4.3.1.3 Ontario Highway Bridge Evaluation Loads

The load-carrying capacity evaluation of an existing bridge may lead to the posting of the bridge with a sign restricting the maximum gross weight of trucks using the bridge. In the past, such signs have shown a single weight limit for all trucks. Single weight limits are restrictive for longer trucks, especially on short-span bridges.

Under the system adopted in this Code, bridges are evaluated for three "levels" [8]. Level 1 applies to the operation of single unit trucks. Level 2 applies to trucks with one trailer or semi-trailer, and Level 3 to truck trains.

Each Evaluation Level load is an upperbound representation of the heavily-loaded trucks in its category.

C2 — 4.3.1.4 Design Lanes

During a bridge lifetime, the number, position and direction of travel of actual traffic lanes may change, depending upon the highway use. The bridge should be capable of supporting the maximum number of loaded lanes that can be accommodated on the bridge within the full deck width. Therefore, the design lanes are derived from the deck width without regard to the way in which lanes may be marked for traffic use.

Provisions for design lanes were adopted from AASHTO Standards [9].

C2 — 4.3.1.5 and C2 — 4.3.1.6 To properly model actual loads, Truck axles and uniformly-distributed loads are applied only where they increase the total load effect.

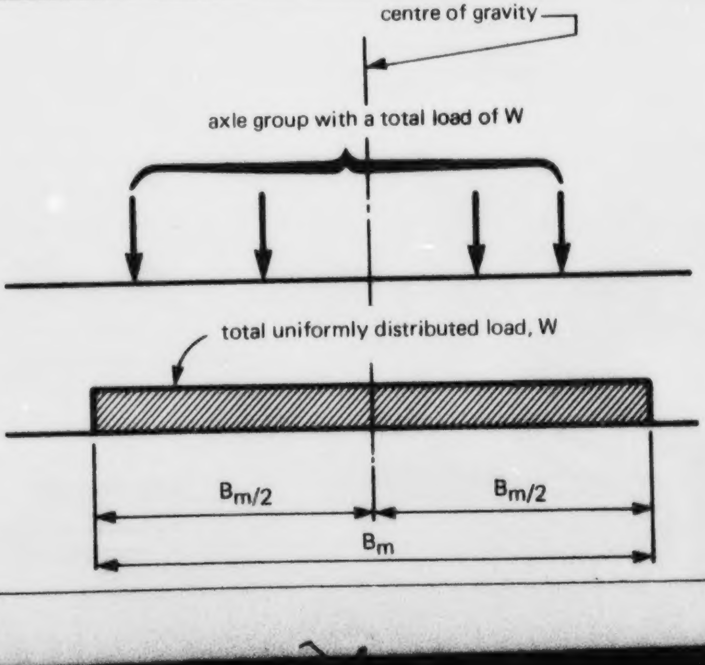

centre of gravity
axle group with a total load of W
total uniformly distributed load, W
$B_{m/2}$
$B_{m/2}$
B_m

Figure **Equivalent Base Length Concept** **C2 — 4.3.1.2(d)**

C2 — 4.3.1.7

Serviceability limit states Type I are concerned with fatigue in structural steel and human response to bridge vibration. For both these limit states, the mean largest effect in a five minute period is considered to be appropriate. This corresponds to a load level close to the legal load limit in an average truck train, which is approximately equal to 80% of one OHBD Truck, located at the centre of the travelled lane.

For simplicity in application, the live load effect for serviceability limit states Type I is determined by placing one OHBD Truck at the critical locations on the bridge. The live load effect is then reduced by the application of a live load factor in accordance with Clause 2—5.1.

C2 — 4.3.1.8

For simplicity in application, the loads for serviceability limit states Type II and for the ultimate limit states are the same. The load effects for serviceability limit states Type II are then reduced by factoring in accordance with Clause 2—5.1.

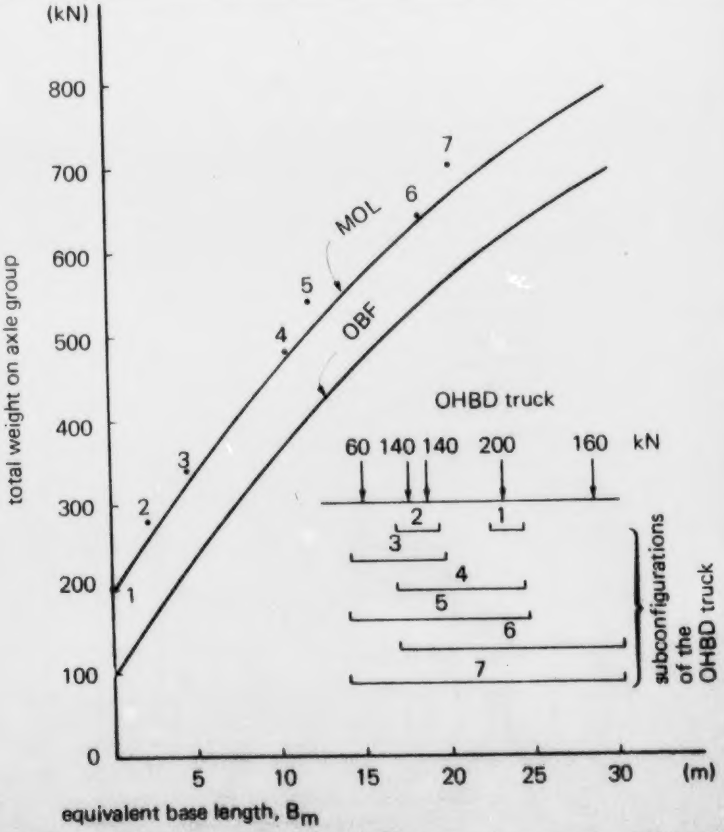
C2 — 4.3.1.9

The modification factors for multi-lane loading as prescribed, apply only to highway live loads. This, however, also causes a reduction in the dynamic load allowance, since the latter, although defined as a load, is expressed as a fraction of the highway live load.

The flow of traffic in a lane is not independent of the presence of vehicles in other lanes. Correlation of the traffic flow in adjacent lanes is affected by a number of parameters including number of lanes, traffic volume, speed, traffic mix and accident situations. The reduced probability of more than one lane being critically loaded at the same time is accounted for by a reduction factor, m_l, for multi-lane loading [4] given by:

$$m_l = \lambda + \frac{1-\lambda}{\Sigma L_k}\sqrt{\Sigma L_k^2}$$

Figure **Comparison of OHBD Truck with MOL and OBF Levels** **C2 — 4.3.1.2(e)**



where λ is a constant which is a function of correlation of traffic flow in adjacent lanes and L_k is the live load effect due to loads in the k^{th} lane. The summation is carried out for the desired number of lanes. For most conditions of traffic flow, the value of λ varies from 0.5 to 0.7.

The specified values of the modification factor were calculated from the reduction factor m_l and a reduction factor applicable to the dynamic load allowance when more than one lane is loaded. The dynamic load allowance reduction factor was derived by assuming that the mean value of the dynamic effect of loading several design lanes is no larger than the square root of the sum of the squares of the contributing effects. This assumption is made frequently in the analysis of the dynamic response of structures and is valid when the contributing effects are uncorrelated.

For most cases, the maximum effect will be obtained by placing either Trucks or Lane Loads in all of the loaded design lanes. Loading of different design lanes simultaneously by Truck and Lane Load need not be considered.

C2—4.3.1.10 Trucks are assumed to remain entirely within their design lanes, as specified in Clause 2—4.3.1.8. This would preclude the consideration of wheel loads closer than 0.6 m to a curb or barrier. In the design of deck slabs and certain components of deck plate and grid systems for which critical load effects are mostly due to one or two wheel loads and increase rapidly as the wheels approach the curb or barrier, it is prudent to consider the wheels in an extreme position, such that they are almost in contact with the curb or barrier and yet are still in motion to the extent that the dynamic load allowance applies. This would be a rare event and is therefore considered only at the ultimate limit states. The second and third axles are specified because their wheels model an actual wheel configuration more realistically than those of the heavier axles.

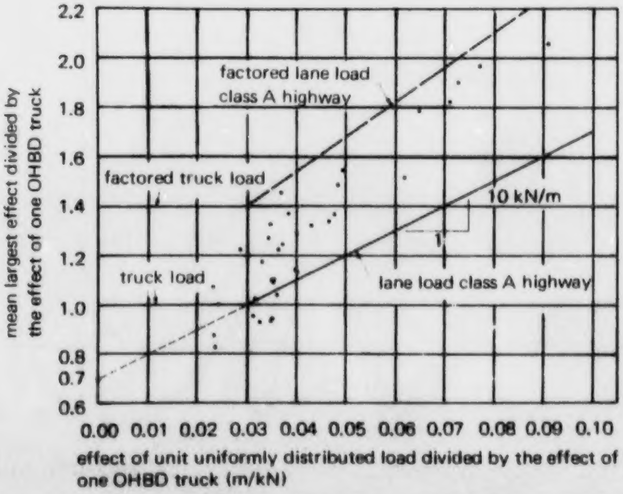
C2—4.3.1.11 Situations in which the wheels of a heavily-loaded truck accidently ride over the sidewalk may be critical for the sidewalk and the structural components supporting the sidewalk. Such events are rare and generally do not involve very heavy trucks, which permits a reduction in the axle loads assumed.

C2—4.3.2 **Dynamic Load Allowance**

The provisions of Clause 2—4.3.2 are based on evidence obtained from field measurement and analysis, tempered with experience, and attempt to reflect the physical process of vehicle-bridge interaction. Extensive background material relevant to Clause 2—4.3.2 is given in the cited references. These references include discussion of the calculation of natural frequencies of bridge superstructures, vehicle and bridge dynamic interaction, damping, and human response to motion.

The term "dynamic load allowance" is used to reflect the various sources of dynamic loading, including discrete and random irregularity of the riding surface, bridge

Figure **Mean Largest Effects by Davenport and Harman Using Trucks from 1975 Survey** **C2—4.3.1.2(f)**



static and vibratory deflections, and the dynamic effects of interaction between a moving vehicle and the bridge. The term "impact factor" has been discarded as its literal interpretation is too narrow to be descriptive of the phenomenon of dynamic loading.

The complex interaction between a bridge and traversing vehicles has been described in other codes [9, 10] by the following simple expression developed in the late 1920s:

$I = 15/(L+38)$

where: I = impact fraction, not to exceed 0.30
L = span length in metres

The allowance for dynamic, vibratory or impact effects is applied as a fraction of the live load stress in a component and for some components can be taken as zero. Confusion can occur as to which component should have the static effects of live load stress amplified for design purposes. The expression for impact fraction is not representative of the response of bridges having fundamental frequencies in the 2 to 5 Hz range [11-14] under the loading of modern heavy commercial vehicles. Further, loads and load factors in this Code differ from those in other codes.

For convenience in design, and continuing current practice, the dynamic load allowance is described in terms of an equivalent static load which is a fraction of the OHBD Load. Thus, the force effects of dynamic load due to the gravity portion of the OHBD Load can easily be calculated. Also, the effect of load on a member or component may be found without the designer having to exercise judgement as to whether the dynamic allowance should be included for that member or component.

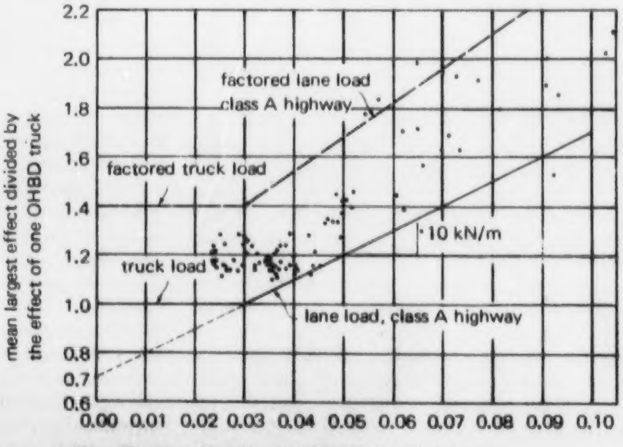
The allowance could also be described in terms of the mass distribution and modes of vibration of a structure. (Such a description must await future editions of the Code; however, preliminary details are available in Reference 1.)

Dynamic effects of load are associated with all parts of the structure where force effects due to the gravity portion of moving loads may be present, including sidewalks, bearings, and substructures. Buried structures are also considered.

C2—4.3.2.1 The dynamic load allowance is an equivalent static load, expressed as a function of the OHBD Load or controlled vehicle load, which is considered for design purposes to be equivalent to the dynamic and vibratory effects of the interaction of the moving vehicle and the bridge. Dynamic load allowance is not required for centrifugal, braking, collision or pedestrian loads. The maintenance vehicle load, Clause 2—4.3.10.3, as specified includes an allowance for dynamic effects.

C2—4.3.2.2 The use of advanced dynamic analyses, or tests, either on structural models or existing structures, is subject to approval. Generally such analyses or test results are not

Figure **Mean Largest Effects Calculated Using the Eight Heaviest Trucks from 1975 Survey** **C2—4.3.1.2(g)**



readily available for design or evaluation use. However, where such analyses or test results are used, the observed data must be calibrated for use in design or evaluation to obtain an appropriate dynamic load allowance [14].

C2 — 4.3.2.3 to C2 — 4.3.2.7

Dynamic load is caused by a combination of bumps in the riding surface or expansion joints resulting in direct impact applied to the bridge deck, dynamic variation in axle loads due to undulation and roughness in the riding surface, and dynamic response of the main longitudinal bridge components to the moving vehicle loads.

The effects of these factors are generally not independent, but the relative contribution from each may vary significantly depending upon the component loaded. The dynamic load allowance for a single truck, or part thereof, is specified in three categories. A smaller allowance is prescribed for the uniformly-distributed load representing traffic in addition to a single truck.

The dynamic load allowance for highway live load is specified in a series of five clauses as follows:

a) loading by a single wheel or axle load (Clause 2 — 4.3.2.3);
b) loading by two to four wheel or axle loads on spans up to 22 m; and loading for cases not specifically referred to in other clauses (Clause 2 — 4.3.2.7);
c) loading by the Truck or the Truck component of lane load for spans greater than 22 m (Clause 2 — 4.3.2.4);
d) loading by the uniformly-distributed portion of lane load (Clause 2 — 4.3.2.5); and
e) loading by the loads generally controlling soil-steel structure design (Clause 2 — 4.3.2.6).

In the Code, the sequence (a) to (e) has been rearranged for clarity of application of clauses.

Clause 2 — 4.3.2.3 covers design of components governed by a single axle unit, or part thereof, which may include slabs, such as concrete deck slabs, pan fill floors, steel orthotropic decks and short-span supporting elements. Dynamic variation in axle load due to roughness in the riding surface, and impact at bumps, is directly transferred to the component, which results in a high dynamic load. The dynamic interaction of such components with the moving load is generally very small, because vibration of the component has a short period compared to the duration of loading.

The specified value of dynamic load allowance is based on test results [15, 18, 19], and assumes the approach riding surface and bridge deck have been paved to acceptable standards. If the approach is unlikely to be paved for an extended period of time, or if expansion joints between the superstructure and approach pavement are not generally flush with the roadway riding surface, vehicles will enter the bridge in a state of excitation. To allow for this, it is recommended that the dynamic load allowance be increased from 0.4 to 0.5 within 3 m or one-tenth of the span length, whichever is greater, from the location of the joint. This distance corresponds approximately to one half cycle of the axle spring for a vehicle travelling at approximately 100 km/h.

Where suspended spans are incorporated in semi-continuous or continuous structures, and also where expansion joints are present within the length of the structure, an increase in dynamic load allowance from 0.4 to 0.5 within 3 m or one-tenth of the span length, whichever is greater, from the location of the joint is recommended.

Appreciable reduction in the dynamic effects of axle load can be achieved by minimizing undulations, pot holes and bumps, especially at or close to expansion joints. Repair of the approaches and deck should be considered as part of any rehabilitation of a structure.

Clause 2 — 4.3.2.4 covers longitudinal components whose design is normally governed by truck or lane load.

A frequency match with typical vehicle suspensions may occur and the effect of axle load impact is not the main dynamic load. The duration of loading is long, so that the bridge and vehicle may interact for several cycles of vibration of the bridge frequency.

Tests have shown that structures with spans greater than approximately 20 m usually have a lower damping ratio than shorter spans. Vibration therefore persists longer and vehicles have a greater probability of entering a bridge already vibrating due to passage of a preceding vehicle. There is increased dynamic response of bridge superstructures having natural frequencies in the range 2 to 5 Hz, a range typical of the bounce frequencies of vehicles [11, 14]. The increase is because of interaction between the vehicle and bridge, and has many of the characteristics of resonance in a simple oscillator. This is shown by the analytical model of a moving pulsating load on a simple span [20, 21].

The complex interaction response of the vehicle and bridge to the vehicle crossing is related to the riding surface roughness, the frequencies of both, and many other factors [14]. The provisions of Clause 2—4.3.2.4 assume that the first longitudinal flexural frequency of the main longitudinal components is an appropriate reference for the dynamic response of the superstructure to vehicle load. Examples are available where, under the action of traffic, superstructures vibrate in higher longitudinal flexural modes, torsional modes, transverse flexural modes, a frequency associated with the crossing vehicle, or some combination of these [11,13,14,22]. However, calculation of higher frequencies as part of bridge design is not warranted at this time.

The clause considers structures where coincidence of vehicle and bridge frequencies is possible, and also where it is unlikely. Where coincidence of frequency between the bridge and a single vehicle is possible, a higher dynamic load allowance is proposed for a frequency range of 2.5 to 4.5 Hz. The dynamic load allowance of 0.40 provides for the quasi-resonance effect, and also for the duration of vibration of structures with low damping.

The values given in Clauses 2—4.3.2.4 and 2—4.3.2.7 were developed from tests [11, 13, 14, 16, 23] using identical values of load factor for live load and dynamic load, and a mean-to-specified value of 0.5 to 0.6. This ratio of 0.5 to 0.6 might appear unusual in a limit states design code. If mean dynamic load allowance values were specified for design, the corresponding load factor would be approximately 2.5. Mean values of the dynamic component of response are approximately 0.20 of the static component of response for structures with frequencies in the 2.5 to 4.5 Hz quasi-resonance range, and about 0.10 to 0.15 for structures outside this range. Coefficients of variation of the dynamic component of 0.6 to 1.0 are typical for the observed data [16].

Design is an iterative process, and application of Clause 2—4.3.2.6 would require a knowledge of the first flexural frequency of the structure in order to determine an initial dynamic load allowance for loading in a single design lane. The following dynamic load allowances are appropriate estimates for preliminary design of main members:

- for simple spans between 22.5 m and 60 m — 0.40
- for simple spans greater than 60 m — 0.20
- for continuous bridges with any span between 22.5 m and 60 m — 0.40
- for continuous bridges with any span greater than 60 m — 0.20

For continuous bridges where the allowance of 0.40 is used in preliminary design, economies may result if the distribution of longitudinal flexural stiffness is chosen so that the first flexural frequency lies outside the range of approximately 2 to 5 Hz [14, 22].

The dynamic load allowance resulting from the application of Clause 2—4.3.2.4 can change as a design develops and improvements to the value of calculated first flexural frequency are made. It is not the intent of Clause 2—4.3.2.4 to force a series of design cycles so as to converge on a value of frequency that is identical at the end of one design cycle to the value at the end of the previous design cycle.

As a guide to designers, the following are suggested:

1) If an assumed or calculated first flexural frequency is within 5% of 2.5 Hz or 4.5 Hz, use a dynamic load allowance of 0.4 in a design cycle.
2) If, using the calculated frequency at the end of one design cycle, the dynamic load allowance (Clause 2—4.3.2.4) is no more than 0.02 greater than that at the end of the previous design cycle, a further cycle to improve the dynamic load allowance is not necessary.
3) If the dynamic load allowance decreases by 0.02 as a consequence of a second design cycle, consideration should be given to the use of the smaller value of the allowance.

The calculation of the first longitudinal flexural frequency of a bridge superstructure should be based upon the actual mass distribution. This includes all structural and non-structural components, and superimposed dead load. Stiffness should be determined using the full cross-section of the superstructure. Composite action should be assumed, even though not considered in the calculation of sectional resistance. Calculations should be based on design values for member sizes, using static values of the moduli of elasticity. An average value of $E_{0.50}$ may be taken for wood. Non-structural components such as curbs and barrier walls, with or without joints, contribute to the stiffness of main longitudinal components. Bracing appears to have little effect on natural frequencies of longitudinal flexural modes.

Clause 2—4.3.2.4 applies only to structures with spans greater than 22 m. This span length was chosen to provide a transition between cases of little dynamic interaction between vehicle and superstructure and increasing dynamic interaction—that is, the effects modelled by Clauses 2—4.3.2.7 and 2—4.3.2.4, respectively.

The transition from Clause 2—4.3.2.7 to Clause 2—4.3.2.4 can be described in terms of either span or frequency. A span of 24.5 m length corresponds to the case where loading from four axles of the OHBD Truck rather than the complete Truck would govern the flexural design of a simple-span beam. A dynamic load allowance of 0.3, as specified in Clause 2—4.3.2.7, corresponds to a flexural frequency of 5.5 Hz, Clause 2—4.3.2.4. The average value of span length corresponding to the average flexural frequency of 5.5 Hz is 22.5 m [16]. To minimize any discontinuity which might result from the application of Clauses 2—4.3.2.4 and 2—4.3.2.7, a span length of 22 m was chosen for use in design based on a frequency consideration and the associated average span.

The lane load represents the OHBD Truck plus additional traffic in the same lane. It is unlikely that all axles of all trucks on the superstructure will be in phase so that a reduced dynamic load allowance, 0.1 rather than 0.2 or more, is prescribed for the trucks additional to the OHBD Truck and simulated by the uniform distributed load. Spans for which Lane Load rather than the Truck controls design will be a function of the dynamic load allowance and, hence, first flexural frequency of the superstructure.

C2—4.3.2.6 The dynamic load allowance specified assumes that design of soil-steel structures is usually based on a single or dual axle loading. The dynamic load allowance value of 0.4 is used for both loading cases to reflect local irregularity in the pavement profile in the vicinity of the crown of the soil-steel structure.

C2—4.3.2.7 Clause 2—4.3.2.7 covers the cases not specifically referred to in Clauses 2—4.3.2.2 to 2—4.3.2.6 and hence acts as a default clause for the dynamic load allowance provisions.

Clause 2—4.3.2.7 mentions specific cases where a dynamic load allowance of 0.3 is appropriate. The dynamic variation in axle loads and the impact due to bumps is similar to the loading case of Clause 2—4.3.2.3, but the increased number of axles (two or more), are not generally in phase so that the dynamic effect is moderated. The dynamic response of a component to moving loads results in some additional dynamic load but the frequencies of a longitudinal component with a span less than 22 m are usually sufficiently high that any frequency match between component and vehicle is unlikely. Similarly for transverse members, the duration of loading is short and a frequency match unlikely.

The increase in the dynamic load allowance of 0.1 recommended for suspended spans and expansion joints within spans may also be applied to Clause 2—4.3.2.7.

C2—4.3.2.8 In design, the force effects transferred from one component to another should include the effects of vibration and impact through the inclusion of the dynamic load allowance as part of such force effects. Designers are advised to transfer the force effects of load, including the dynamic load allowance, to the top of the foundations where earth cover provides damping and reduces the vibratory effects of live load.

Consideration was given to the identification of components for which the dynamic load allowance should be included. Such a component identification scheme was found to be intractable as there is not a universally-accepted definition for all components which uniquely defines a structural function. For example, a diaphragm may connect two girders, transferring deck loads to the girders and thence bearings, or a diaphragm may connect two girders and support both via a bearing. In each case the nature of the force effects introduced into a similar component differs markedly.

Clause 2—4.3.2.8 results from the dynamic load allowance being load-, span- and frequency-based, rather than component- and increased-stress-based.

C2—4.3.2.9 This clause is based on existing practice [9, 10]. As the depth of earth cover increases from zero to 2.0 m, a reduction in the dynamic load allowance is applied. Recent field studies of soil-steel structures having various depths of cover from the paved riding surface to the top of the structure indicate that the dynamic effects of load decrease with depth [17]. A dynamic load allowance of 0.10 is appropriate for soil-steel structures for depths of cover in excess of 2 m because the soil mass has a high energy absorption capacity.

C2 — 4.3.2.10

The wood components considered in this clause include bridges fabricated largely of wood, composite concrete and wood, and transversely- or longitudinally-laminated wood decks.

Wood bridges are usually short-span structures and the duration of loading is short. The impactive effects of wheel load on the deck element will be present. In addition, for longer span structures, interaction between the vehicle and bridge is possible.

The dynamic load allowance has been reduced for wood in recognition of improved response to load applied quickly (short-span structures) and of higher damping (long-span structures). For wood structures supporting earth cover, a minimum allowance of 0.07 would apply.

C2 — 4.3.2.11

This clause considers cases where it is necessary to evaluate the effects of a single vehicle with an exceptionally heavy load, or unusual axle spacing, or both, crossing a structure at a controlled speed under escort. A special static analysis of the load distribution characteristics of the structure is generally required to permit the passage of such a vehicle. The clause does not apply where speed control cannot be ensured and where other trucks can be present on the structure.

The reductions are based on the concept that dynamic effects decrease with vehicle speed [12, 15], and approach zero when the vehicle moves across the bridge at crawl speed. Dynamic effects per unit load also decrease as additional axles are introduced to permit increased gross vehicle load [12, 14, 16]. The reduction assumes that the vehicle moves smoothly across the bridge and that the sudden application of brakes is extremely unlikely.

C2 — 4.3.3 **Sidewalk Load**

C2 — 4.3.3.1

The pedestrian live load of 5.0 kPa allows for a crowd of spectators standing close together on a walkway area. The same load value is specified in the British Standard [24] and is close to the value specified for corridors in the National Building Code and in CSA Standard S-6 [10 and 25].

The smaller maximum value of 4.0 kPa for pedestrian live load is used for structural components which support both pedestrians and the highway live loads because it is improbable that the sidewalk would be crowded at the same time as the traffic lanes are critically loaded with heavy vehicles. For greater loaded lengths, the sidewalk load is further reduced because of the reduced probability of crowding along the full length.

The intensity of the reduced sidewalk load depends upon the total loaded length of the sidewalk. Where two or more loaded segments of the sidewalk are separated by unloaded segments, the loaded length is taken to be the sum of the lengths of all the loaded segments. Each segment is loaded by the load intensity based on the loaded length so determined. Figure C2 — 4.3.3.1 illustrates such cases.

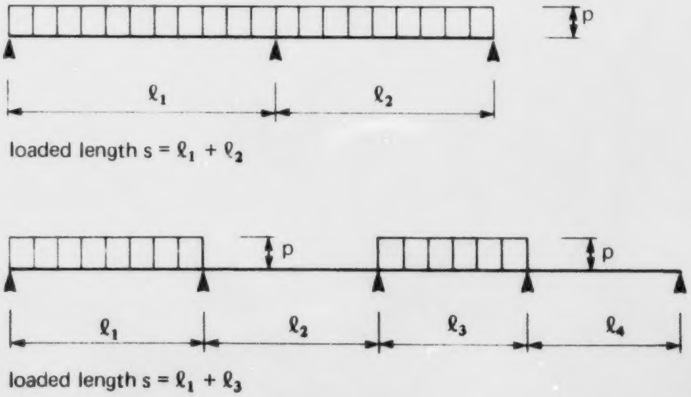
C2 — 4.3.3.2

The loads described are considered to be mutually exclusive.

C2 — 4.3.4 **Curb Load**

The loads of 25 kN/m and 40 kN are derived from values given in CSA Standard S6-1974 [10] multiplied by a factor of 1.75. This factor was determined by considering curb resistance at the ultimate limit state and at the working stress level and the vari-

Figure **Loaded Length for Pedestrian Load on Sidewalk** **C2 — 4.3.3.1**



loaded length $s = \ell_1 + \ell_2$

loaded length $s = \ell_1 + \ell_3$

ous load and performance factors for each method of design. Thus, the specified curb load in this Code will result in designs consistent with CSA Standard S6-1974.

C2—4.3.5 **Railing Load**

C2—4.3.5.1 The pedestrian load on railings is based on the requirements for railings for exits and stairs in the National Building Code of Canada [25].

C2—4.3.5.2 The specified traffic load for railings is based on the lateral impact force of a passenger vehicle hitting the railing. Table C2—4.3.5.2 summarizes the calculated ultimate lateral impact force due to the passenger vehicle and also due to various other types of vehicles [26] for comparison. The lateral impact force is distributed over the area of impact. An equivalent static concentrated lateral force that causes the same effect would be about 80% of the distributed force given in Table C2—4.3.5.2. Considering the load factor of 1.40 specified in the Code, the specified load value of 80 kN is determined.

Table **Maximum Lateral Impact Force Due to Different Types of Vehicles** **C2—4.3.5.2**

Vehicle, Gross Load	Maximum Lateral Impact Force (kN)
Passenger Vehicle, 20 kN	140
School Bus, 90 kN	370
Commercial Bus, 180 kN	670
Concrete Mixer Truck, 315 kN	1110
Heavy Truck, 450 kN	1560

C2—4.3.5.4 It is not likely that the curb and the railing would be struck simultaneously.

C2—4.3.6 **Pedestrian Bridge Load**

C2—4.3.6.1 Clause C2—4.3.3.1 describes how the loaded length should be determined.

The loading of 5.0 kPa allows for a crowd of spectators standing close together on the walkway area. For loaded lengths over 30 m, the loading is reduced uniformly to a minimum value of 2.0 kPa at a loaded length of 105 m, because of the reduced probability of crowding along the full length.

C2—4.3.6.2 The loads described are considered to be mutually exclusive.

C2—4.3.7 **Centrifugal Force**

The magnitude of the centrifugal force is proportional to the square of the vehicle speed. Therefore, the critical centrifugal force occurs in free traffic flow at high speed. In free traffic flow, vehicles tend to maintain a clear distance of at least 1m for every 2 km/h of vehicle speed. Therefore, for conventional structures it is unnecessary to consider more than one truck in a lane for critical centrifugal forces. For a very long curved structure in which transverse forces from several spans are resisted at one location, special consideration is appropriate. It will probably be found, however, that in such cases, earthquake loads control the design even if more than one truck per lane is considered.

C2—4.3.8 **Braking Force**

C2—4.3.8.1 While braking, vehicles apply a transient longitudinal force to the top of the bridge deck. This force causes a longitudinal motion of the superstructure which is restrained by support reactions. The magnitude of the restraining forces depends upon the dynamic characteristics of the bridge superstructure, the vehicle, and the bridge bearings and piers. For simplicity in design, the braking force in this Code is specified as an equivalent static longitudinal force that will cause load effects similar to those caused by braking vehicles.

The vehicle braking force is a function of the coefficient of friction between the tires and the wearing surface. The coefficient of friction depends on the skid resistance qualities of the wearing surface. Under extreme conditions, the dynamic braking force due to an axle may be as much as 80% of the axle load [27]. However, the peak value of braking force as a percentage of gross vehicle weight is smaller since all axles do not exert maximum braking force simultaneously. A theoretical consideration of dynamic behaviour for typical structures suggests that although the equivalent static force should be substantially smaller than the peak force exerted by vehicles, it is still quite large. No evidence can be found confirming the occurrence of such large forces and the apparent absence of damage among structures designed to earlier codes, prescribing braking forces as low as 5% of vehicle weight, cannot be ignored. The equivalent static forces prescribed are considered to be a reasonable compromise, representing a substantial increase over values previously prescribed but not being unduly conservative in comparison with the forces that can theoretically arise. Special consideration should be given to structures that are unduly susceptible to longitudinal deck forces.

C2 — 4.3.8.2

The height specified corresponds to the centre of mass of a loaded truck.

Braking force for multi-lane loading is affected by two considerations:

1/ The reduced probability of having critical highway live load in more than one lane at the same time.

2/ The reduced probability of vehicles in all loaded lanes braking simultaneously.

Approximate values of the factors to account for both considerations are given in Table C2 — 4.3.8.2. These modification factors suggest that the braking force in three or more lanes is no more critical than in two lanes and, therefore, need not be considered.

Table **Modification Factors for Braking Force** **C2 — 4.3.8.2**

Number of Loaded Design Lanes	Modification Factor
1	1.00
2	0.75
3	0.50
4	0.35

C2 — 4.3.10 **Maintenance Access Loads**

C2 — 4.3.10.1 and
C2 — 4.3.10.2
These loads have been used for some time by the Ministry.

C2 — 4.3.10.3
Where the width of a walkway and the geometry of its approaches are such that a front-end loader could be used for snow removal, the local effect of its wheel loads may be critical. For pedestrian bridges narrower than 3 m, the effect of the pedestrian load exceeds the effect of a small snow-removal vehicle together with the snow on the bridge.

C2 — 4.3.10.4
Maintenance access loads do not occur frequently enough to cause fatigue but, for pedestrian bridges, the possibility of kinking or cracking should be considered.

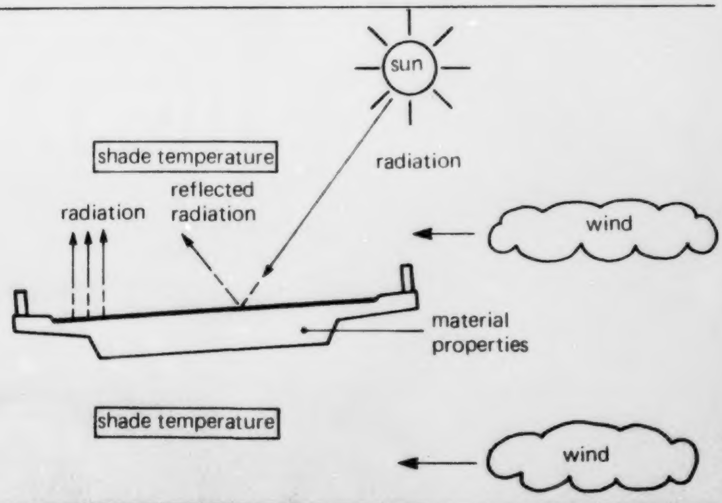
Thermal, Shrinkage and Creep Effects **C2 — 4.4**

C2 — 4.4.1 **General**

Traditionally, bridges have been designed to accommodate or resist only the overall longitudinal movement arising from temperature strain. Expansion joints and bearings have been provided to allow for movement. Bridges do, in fact, react in other ways to change in climatic conditions and instances of damage have occurred when adequate provision for movement has not been made [28, 29].

Clause 2 — 4.4 provides detailed guidance for the calculation of overall longitudinal movements by specifying a range of effective bridge temperatures, which depend on location and the form of construction. The specified range of effective temperatures represents the average range to be considered in design. Factors such as altitude, exposure of the structure and orientation to the sun may result in localized effective temperatures which are beyond the specified range.

Figure **Factors Affecting Thermal Response of Superstructure** **C2 — 4.4.1**



Temperature changes within a bridge superstructure are a function of the shade temperature, intensity of incident solar radiation, the absorptivity of the superstructure materials, and the depth of the superstructure [30], as shown in Figure C2—4.4.1. During periods of hot, sunny weather, the effective temperature of a bridge superstructure can exceed the maximum shade temperature because of incident solar radiation.

Similarly, during periods of cold weather, the effective temperature of a bridge superstructure can be below the minimum shade temperature because of reradiation [31]. The thermal conductivity of concrete is relatively poor and temperature gradients can occur through the depth of concrete superstructures. Thermal gradients are discussed in more detail in C2—4.4.3.4.

The statement concerning wood structures is based on observations. Conventional short-span wood structures appear to perform satisfactorily, even when there are a number of spans, with no particular provision for temperature movement. Positive constraints, which could cause forces for which the structure is not designed, should nevertheless be avoided.

The specifications for shrinkage and creep constitute approximate predictions based on typical materials and conditions. If a precision of plus or minus 40% is unsuitable, methods must be developed based on representative testing.

C2—4.4.2 Movements and Load Effects

The plan geometry of a bridge superstructure should be considered when assessing the response of the structure to changes in effective temperature. Bridges which are straight in plan will generally exhibit movements along their centreline when subjected to changes in effective temperature. Bridges which are curved in plan will have two components of movement; a longitudinal component along the centreline of the superstructure, and a transverse component perpendicular to the longitudinal centreline. These movements are shown in Figure C2—4.4.2(a).

Bridge geometry will influence the design of expansion devices at abutments. With structures curved in plan or with skew joints, the relative movement of the superstructure may or may not be normal to the line of the expansion joint, as shown in Figure C2—4.4.2(b). This can lead to binding of expansion devices and to high transverse forces on joints, bearing and abutments, with corresponding reactions on intermediate piers. It is, therefore, important to assess this transverse movement and to design the joints, bearings and structure accordingly.

To determine the actual movement of a bridge superstructure, the position of the neutral or stationary point must be established. The stationary point is that point about which a given set of movements takes place. The position of the stationary point depends upon the geometry of the superstructure, the location of the piers and bearings and their relative shear stiffnesses, and the type of dimensional change. For straight structures, the stationary point is generally located on a longitudinal cen-

Table C2—4.4.4.2

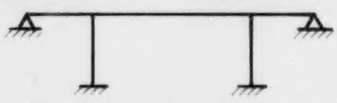
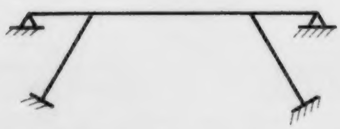
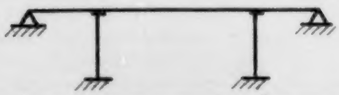
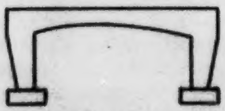
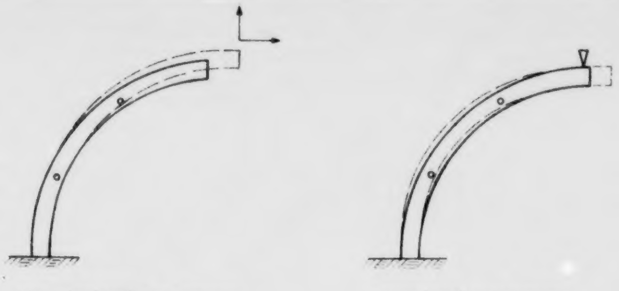
Structural Form	Load Effects
	Column Moments Deck Moments Axial Force in Deck
	Column Moments Axial Force in Columns Deck Moments Axial Force in Deck
	Column Moments Axial Force in Deck
	Moments in Deck and Legs Axial Force in Deck

Figure **Response of Curved Structures** **C2 — 4.4.2(a)**



unrestrained movement of curved structure

restrained movement of curved structure

Figure **Normal and Transverse Displacement Across Skew Joint** **C2 — 4.4.2(b)**

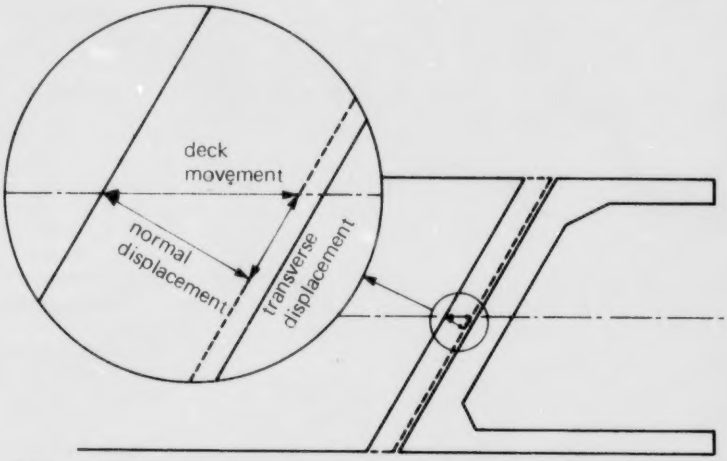
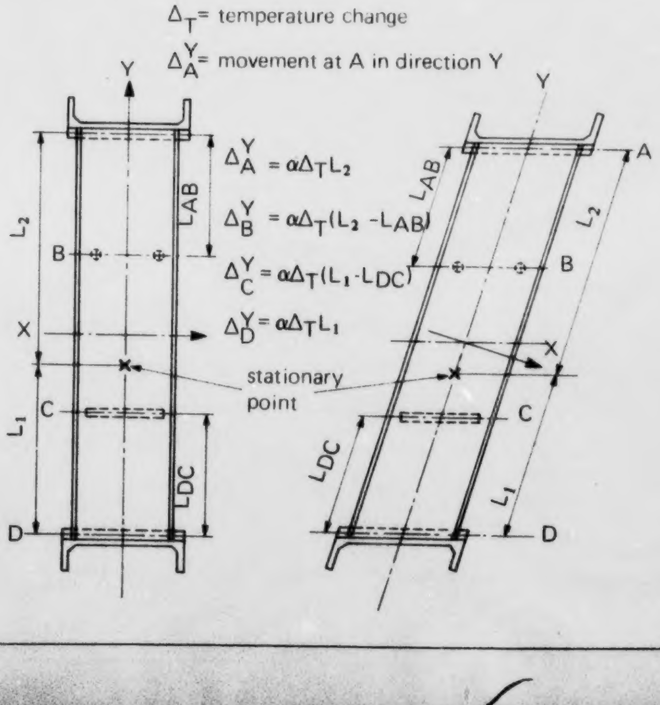

deck movement
normal displacement
transverse displacement

Figure **Stationary Point in Straight and Skewed Superstructure** **C2 — 4.4.2(c)**


Δ_T = temperature change
Δ_A^Y = movement at A in direction Y
$\Delta_A^Y = \alpha\Delta_T L_2$
$\Delta_B^Y = \alpha\Delta_T(L_2 - L_{AB})$
$\Delta_C^Y = \alpha\Delta_T(L_1 - L_{DC})$
$\Delta_D^Y = \alpha\Delta_T L_1$
stationary point
Y
X
A
B
C
D
L_2
L_1
LAB
LDC

treline of symmetry, as shown in Figure C2—4.4.2(c). In curved structures, there are two components of movement; hence the stationary point will generally not be located on the longitudinal centreline and may lie outside the plan of the superstructure. Movements are towards the stationary point for contraction as shown in Figure C2—4.4.2(d). For a detailed explanation of procedures for determining the location of the stationary point, the reader is directed to Reference 32.

The restraint of thermal movements will give rise to force effects in both the superstructure and substructure. Even in "unrestrained" structures, loads will develop as a result of friction between bearing plates or shear stiffness of elastomeric bearings. Clause 2—4.4.2 draws this to the attention of the designer. Table C2—4.4.2 indicates the load effects which can develop in different structural forms as a result of restrained thermal movements.

The provisions of Clause 2—4.4.2 are based on the assumption that an elastic analysis is used to assess the forces due to movement. In some cases it may be necessary to consider inelastic action of the structure at the ultimate limit state in order to assess the forces due to movement. When an inelastic distortion is calculated, the section assumed to rotate inelastically must have sufficient rotational ductility to accommodate such movement without loss of strength.

C2—4.4.3 **Temperature Effects**

C2—4.4.3.1 **Temperature Range**

The range of effective temperatures given in Clause 2—4.4.3.1 reflects the thermal properties of the constituent materials of the bridge superstructure. Because of the relatively high conductivity of steel, steel structures exhibit a larger range in effective temperature than do concrete structures [31].

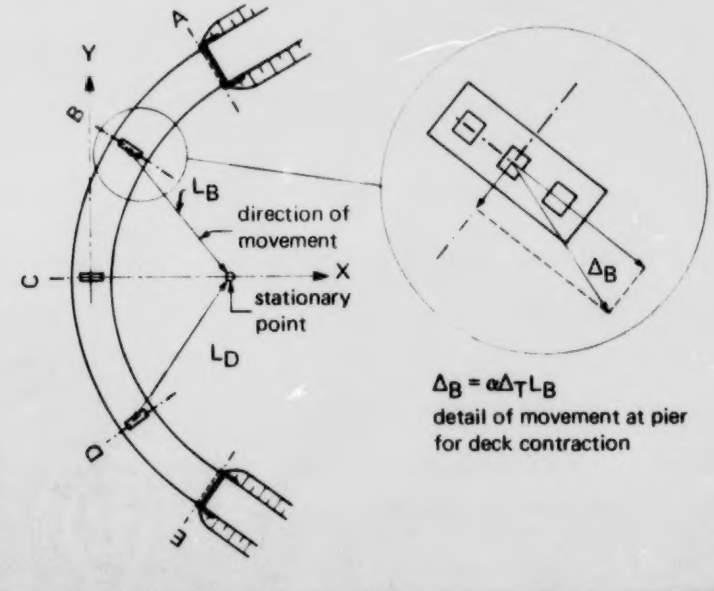
In concrete structures (i.e., Type C), which are deep, only the upper and lower surfaces of the bridge superstructure are affected by rapid changes in climatic conditions. The central portion of deep concrete structures is influenced by long-term variations in mean daily temperatures but not by diurnal heating cycles [33]. This permits reductions to the effective temperature range with increasing depth. Permissible, depth-dependent reductions are assumed to be similar for Type A and B structures.

Asphalt on a bridge deck increases solar radiation effects but also provides insulation. The provisions of Clause 2—4.4.3.1 are applicable to bridges with or without asphalt.

C2—4.4.3.2 **Effective Construction Temperature**

The temperature given is an approximate average effective temperature. It is assumed that bridges are not constructed in very cold conditions without protection.

Figure **Stationary Point In Curved Structure** **C2—4.4.2(d)**



$\Delta_B = \alpha\Delta_T L_B$

detail of movement at pier for deck contraction

When thick concrete members are cast-in-place, there is a considerable rise in temperature brought about by the chemical processes involved in the setting and hardening of the cement. The heat liberated is known as the heat of hydration. This heat dissipates over a period of a few days, depending on the size of the concrete member. An idealized time-temperature graph is shown in Figure C2—4.4.3.2, which is based on temperature readings taken during construction of a number of bridges. During the casting of a bridge superstructure, an initial set would occur before the maximum temperature is reached, but until concrete has begun to set around bearings there is still some freedom to expand. For this reason, the temperature at the time the superstructure becomes effectively attached to the bearings or the substructure is difficult to determine. A further complication when elastomeric bearings without dowels are used, is that the bearings may slide, up to the time of load transfer caused by falsework removal or prestressing. Observations at a number of bridge sites, after casting and prior to post-tensioning, indicate that superstructures contract and cause an amount of bearing movement or distortion consistent with the assumption that initial hardening of the concrete occurs at a temperature between the ambient temperature and the peak temperature. For the design of bearings and expansion joints, the subsequent contraction should be considered and a temperature drop of 25°C is recommended for the design of normal types of structures if a more precise prediction cannot be made.

C2—4.4.3.3 Position of Bearings and Expansion Joints

This is a precautionary cross-reference.

C2—4.4.3.4 Thermal Gradient Effects

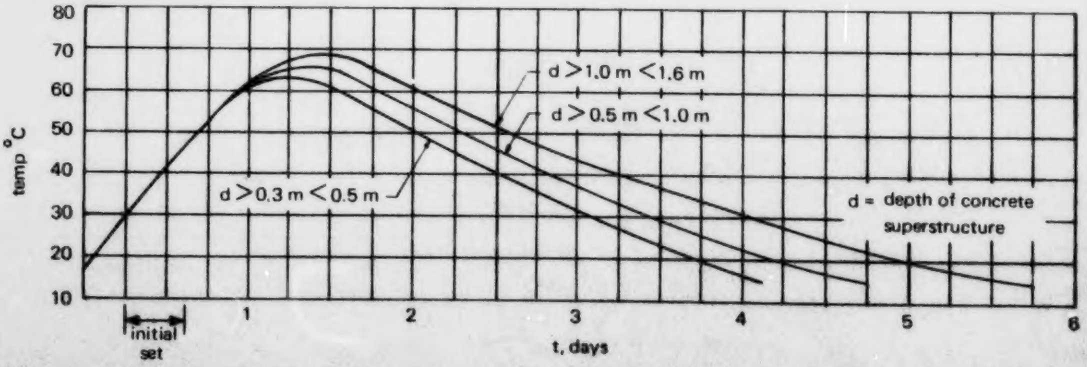
Rapid temperature variations on the surface of a bridge superstructure will result in a non-uniform temperature distribution through the depth of the superstructure. The magnitude of the temperature gradient between any two points will be a function of the thermal properties and the geometry of the bridge superstructure.

Thermal gradients will generally take two forms; linear or non-linear. Shallow Type A structures will, because of the high thermal conductivity of steel, exhibit generally small linear gradients. In deep Type A structures, non-linear gradients will be present [34].

In composite superstructures (i.e., Type B) there is generally a linear thermal gradient in the concrete deck [35]. The temperature of the steel portion of the superstructure can, however, be different from that at the top of the concrete deck. The underside of the deck and the steel will generally assume the same temperature, corresponding to the local shade temperature.

Because of the poor thermal conductivity of concrete, large gradients in concrete superstructures (Type C) can occur [36]. In shallow sections, i.e., less than 0.3 m deep, temperature gradients are nearly linear while in deeper sections gradients are non-linear. Sectional curvature results from a linear gradient. Non-linear gradients cause, in addition to curvature, self-compatibility strains which produce no displacements of the structure as a whole, but set up self-equilibrating stresses. If these strains are neglected, an equivalent linear temperature may be used in design, as indicated in Figure C2—4.4.3.4(a).

Figure **Temperature During Hydration and Cooling** **C2—4.4.3.2**



Curvature strain results in upward displacement of the superstructure for positive temperature differentials, and a downward displacement for negative temperature differentials. These displacements, in the presence of restraints, will produce bending moments and associated reactions.

The moments induced in continuous structures as a consequence of restrained curvature strain, shown in Figure C2—4.4.3.4(b), can be determined using the non-dimensional curvature-depth relationships given in Figure 2—4.4.3.4.

Maximum moment, occurring at a support, is given by the expression:

$$M = \frac{CEI}{R}$$

where: R = radius of curvature
EI = longitudinal flexural rigidity
C = 1.5 for the first interior supports of multi-span bridges, 1.0 for remaining interior supports in multi-span bridges, and is a non-dimensional coefficient.

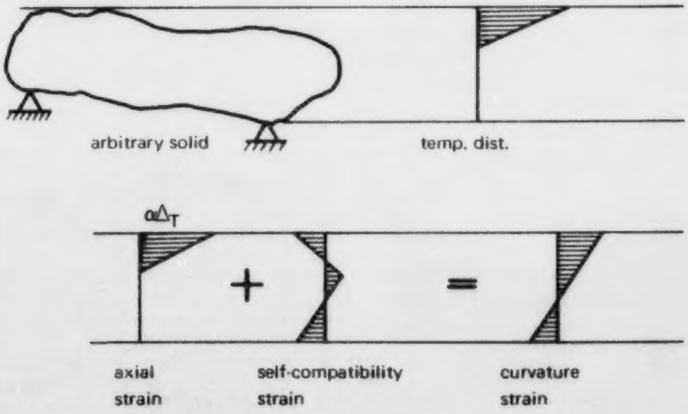
Stresses induced in continuous structures as a result of restrained curvature have led to cracking in Type C bridge superstructures [28].

A temperature rise at the upper surface of a continuous bridge superstructure produces flexural moments which induce tensile stress on the lower surface. In prestressed structures, areas may exist which, under prestress and dead load, have a low compression stress reserve at the bottom surface. These zones are normally near the interior supports. Cracking can result from the combination of low compressive stress reserve and induced tensile stress.

Of particular concern are T-sections, including those formed of pre-cast I-beams with a composite slab. The unfavourable position of the geometric centroid generally results in a relatively low compression stress reserve under dead load and prestress at the underside, and the stress increase for any given temperature gradient is greater than for an I-section or hollow box-section. The stresses resulting from restrained curvature should therefore be added to those due to dead load and prestress throughout the length of the structure. Sufficient non-prestressed reinforcement should be provided in those areas where the allowable tensile stress is exceeded. In those cases where there is a low compression stress reserve, typically T-sections, local prestressing may be provided.

The imposed curvature due to temperature gradients can induce torsional and transverse effects in structures curved in plan or with support locations in an irregular pattern. For example, the structure shown in plan in Figure C2—4.4.3.4(c) will be subjected to additional torsional forces as a result of an imposed curvature and,

Figure **Components of Thermal Strain** **C2—4.4.3.4(a)**



depending upon the magnitude of the dead load reactions and curvature, uplift can occur at the end supports.

C2—4.4.3.5 **Coefficient of Thermal Expansion**

The coefficient of linear thermal expansion for concrete varies due to the composite nature of normal Portland cement concrete, and will depend largely on the type and proportions of aggregate and cement used, the moisture content of the hardened concrete and the method of curing employed.

Table C2—4.4.3.5 shows the coefficient of linear thermal expansion of concrete made with various aggregates and cured under different conditions [37].

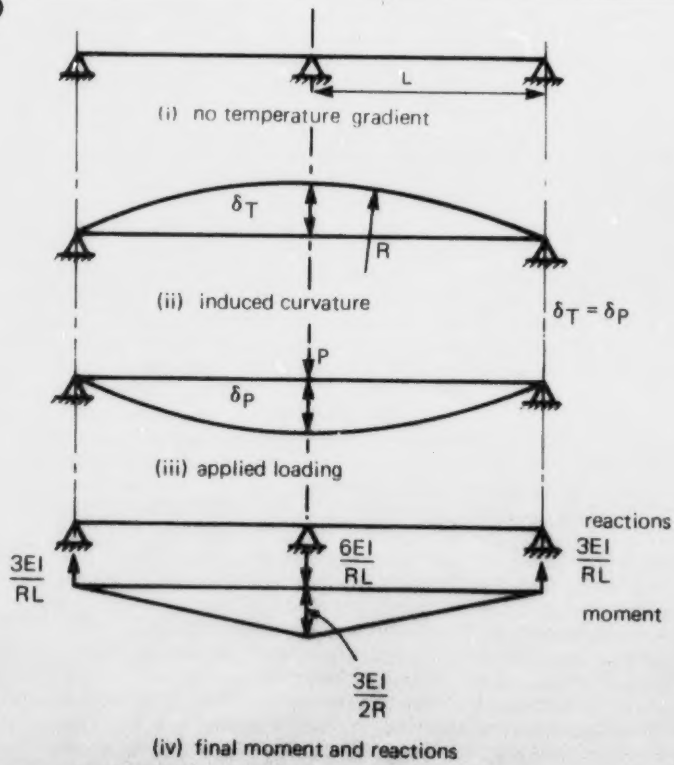
In composite Type B structures, the linear coefficient of thermal expansion is a combination of the thermal coefficients of the constituent materials.

The linear coefficient of expansion for a composite section is given by the following expression [31]:

Table **Coefficient of Linear Thermal Expansion** **C2—4.4.3.5**

Type of Aggregate	Coefficient of Linear Thermal Expansion $\times 10^6$/°C		
	Air-Cured Concrete	Water-Cured Concrete	Air-Cured and Wetted Concete
Gravel	13.1	12.2	11.7
Granite	9.5	8.6	7.7
Quartzite	12.8	12.2	11.7
Dolerite	9.5	8.5	7.9
Sandstone	11.7	10.1	8.6
Limestone	7.4	6.1	5.9
Portland Stone	7.4	6.1	6.5
Blast Furnace Slag	10.6	9.2	8.8
Foamed Slag	12.1	9.2	8.5

Figure **Induced Moments and Reactions** **C2—4.4.3.4(b)**


L
(i) no temperature gradient
δT
R
(ii) induced curvature
δT = δP
P
δP
(iii) applied loading
reactions
3EI/RL
6EI/RL
3EI/RL
moment
3EI/2R
(iv) final moment and reactions

$$\alpha_{comp} = \frac{A_c E_c \alpha_c + A_s E_s \alpha_s}{A_c E_c + A_s E_s}$$

where:
α_{comp} = the thermal coefficient of expansion for composite section
α_c = the thermal coefficient of expansion for concrete
α_s = the thermal coefficient of expansion for steel
A_c = the cross-sectional area of concrete
A_s = the cross-sectional area of steel
E_c = the modulus of elasticity of concrete
E_s = the modulus of elasticity of steel

C2—4.4.4 **Shrinkage in Concrete**

C2—4.4.5 **Creep in Concrete**

The design recommendations are based on Reference 38. For the evaluation of existing structures, Reference 39 should be consulted.

Wind Loads **C2—4.5**

The scope and overall arrangement of the clauses, and some prescriptive provisions, parallel the AASHTO *Standard Specifications for Highway Bridges* [10] *and those for Structural Supports for Highway Signs, Luminaires and Traffic Signals* [40]. The procedures and terminology used to specify the wind loads, however, follow those of the *National Building Code of Canada* [41], referred to hereafter as the NBC. Where the specific sources of prescribed aerodynamic data are not given, the data were arrived at by comparisons with other wind loading codes and wind tunnel test data.

C2—4.5.1 **General**

C2—4.5.1.1

The procedures prescribed in this clause are primarily intended for conventional highway bridges with spans not exceeding 125 m and for various lighting standards, sign and traffic signal supports, various barriers (including noise, visual and other types), and individual structural elements. Although the procedures provided also apply to lighter and more slender bridges such as pedestrian overpasses, the prescriptive material is based on the assumption that the performance of the structure is not sensitive to wind action, i.e., the structural design is not governed by wind loads.

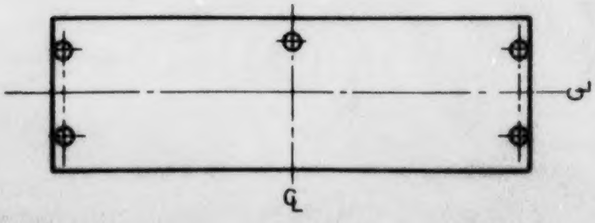
For structures highly sensitive to wind loading, a more detailed analysis of dynamic wind effects based on approved methods is recommended. If significant uncertainties remain, representative wind tunnel tests may be appropriate. Bridge structures are required to resist wind induced horizontal drag and vertical lift loads applied as simultaneously acting equivalent static loads. Following the procedures of the NBC, the magnitudes of these loads are based on a geographically varying mean reference wind pressure increased by a gust effect coefficient to allow for wind induced dynamic effects.

Lighting standards, sign and traffic signal supports, various types of barriers and other slender structural elements are required to withstand the wind induced horizontal drag load, determined following procedures similar to those used for bridge structures and assumed to act as an equivalent static load, as well as the effects of vortex shedding excitation. Vortices shed from these structures produce dynamically acting lateral or across-wind loads. The dynamic magnification of these loads by the structure must be allowed for. Although generally smaller than the drag forces, vortex shedding induced forces may govern the design of some slender, lightly damped structures. In view of their frequent occurrence, the possibility of fatigue damage becomes an important consideration.

C2—4.5.1.2 **Reference Wind Pressure**

The Commentary on the Climatic Data of Appendix A2—1 gives a description of the reference wind pressure, q. The 20% increase in the reference wind pressure to

Figure **Irregular Support Geometry** **C2—4.4.3.4(c)**



account for funnelling is generally conservative. Special consideration, however, is recommended for bridge sites in precipitous terrain or in close proximity to very tall buildings.

C2—4.5.1.3 Gust Effect Coefficient

To allow for dynamic excitation due to buffeting by atmospheric turbulence or "gusting", and a possible dynamic magnification of the structural response due to resonant vibrations, the mean wind load, based on the hourly mean reference pressure, q, for the design return period, is multiplied by the gust effect coefficient, C_g. The load effects due to the static application of the resulting load correspond in magnitude to the expected instantaneous peak load effects associated with the same return period.

Conventional highway bridges with spans not exceeding 125 m are unlikely to experience significant wind induced resonant vibrations and the use of $C_g = 2$ for such structures largely accounts for fluctuating wind loads resulting from turbulent variations about the hourly mean wind speed used to establish q. However, $C_g = 2.5$ is used for light slender structures and for individual structural elements in order to allow for some increase in the effective loading due to possible wind induced resonant vibrations.

Larger values of C_g may be appropriate for structures found to be unusually sensitive to wind loading. A detailed analysis of dynamic action is recommended for such structures. If approved, such procedures may follow methods similar to those outlined in References 41, 42, 43, 44, and 45.

C2—4.5.1.4 Exposure Coefficient

The definition of C_e is in accordance with the NBC "simple procedure". Due to the uncertainty about the vertical variation of the mean wind speed close to the ground, C_e is taken as unity for heights below 10 m. C_e is not very sensitive to the height H, and a rough approximation is adequate. The "top of the structure" may be the top of the vehicular barrier or the top chord of a through or half-through truss. Members and appurtenances that contribute insignificantly to the exposed frontal area should not be considered to affect the height of the structure. A more accurate determination of C_e is justified if the wind load evaluation is based on more detailed methods.

C2—4.5.1.5 Non-Uniform Loading

Wind loads exhibit significant spatial variations and unbalanced loads must be examined in order to determine critical load effects. The degree of load unbalance specified in this clause is an estimate based on the NBC. A more severe form of unbalance may be appropriate in situations where the dynamic component of the wind induced response is found to be pronounced.

C2—4.5.1.7 Alternative Methods

The use of wind tunnel tests is recommended for structures and components of unusual geometry, as well as structures found to be unusually sensitive to wind induced effects evaluated on the basis of normal design procedures; in other words, where structural design is found to be governed by wind loads. Such tests may comprise measurements of mean load coefficients using rigid models or measurements of the mean and dynamic wind induced response using correctly scaled aeroelastic models, which simulate the aerodynamics of the structure as well as its mass, stiffness and damping properties. In both cases it is important to simulate the mean and turbulent characteristics of natural wind in the wind tunnel [46, 47]. Proper allowances for Reynolds number effects should be made, particularly for rounded shapes.

C2—4.5.2 Design of Superstructure

C2—4.5.2.1 General

Wind induced loads for most highway bridges are largest for a wind direction perpendicular to the longitudinal axis of the superstructure. In the case of a curved bridge structure, the longitudinal axis of the superstructure is selected so that wind effects are maximized. Drag induced horizontal loads in Clause 2—4.5.2.2 and lift induced vertical loads in Clause 2—4.5.2.3 specified for the design of the superstructure are thus prescribed only for this critical wind direction. Overturning effects, resulting from wind induced moments about the longitudinal axis are accounted for by applying the vertical load as an equivalent line load at the windward quarter point of the superstructure in Clause 2—4.5.2.3. For bridge types with continuity between the deck structure and the supports, such as rigid frame structures, wind loads acting on the supports, as prescribed in Clause 2—4.5.3, must be included in the design of the structure. In this case, wind directions other than perpendicular to the longitudinal axis of the superstructure should also be examined. In assessments of wind action on the entire structure, the modification factors for the skew angle of wind, given in Table 2—4.5.3.2, apply.

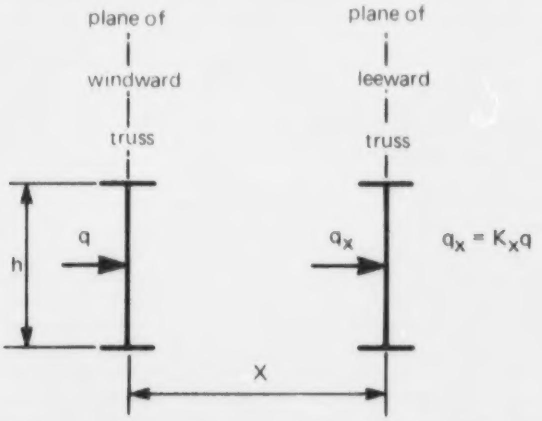
C2 — 4.5.2.2 **Horizontal Drag Load**

Most highway bridge superstructures (typically slab, plate girder, and box girder configurations), act aerodynamically as single bodies. That is, there is no airflow through the superstructure. In such cases, the prescribed horizontal wind load, taken to act on the exposed frontal area, accounts for the total wind induced drag load on the superstructure. This assumption is not valid if there is significant flow through the superstructure, as in the case of truss bridges. In such cases, the wind force on the leeward truss or trusses must be included in the calculation of F_h. Due to shielding, the wind load on the leeward truss diminishes with reduced truss spacing and increased solidity ratio A_s/A. The effect of shielding is given in Table C2 — 4.5.2.2 [41].

Table **Shielding Factors, K_x, for Trusses** **C2 — 4.5.2.2**

X/h	A_s/A						
	0.1	0.2	0.3	0.4	0.5	0.6	0.8
0.5	0.93	0.75	0.56	0.38	0.19	0.00	0.00
1.0	0.99	0.81	0.65	0.48	0.32	0.15	0.15
2.0	1.00	0.87	0.73	0.59	0.44	0.30	0.30
4.0	1.00	0.90	0.78	0.65	0.52	0.40	0.40
6.0	1.00	0.93	0.83	0.72	0.61	0.50	0.50

K_x = shielding factor,
A_s = exposed frontal area of truss,
A = gross area of truss in elevation



Shielding is taken into account by multiplying the reference wind pressure q by K_x for the first leeward truss and all successive leeward trusses.

Unpublished tests at the Boundary Layer Wind Tunnel Laboratory of the University of Western Ontario indicate that $C_h = 2$, based on the entire exposed frontal area, is a conservative value for common slab, plate girder, and box girder superstructures.

For individual components of highway bridge superstructures (e.g., truss elements), the approximate values of C_h given below can be used. For more exact values, reference should be made to Table 2 — 4.5.4.2.

a) For angular shapes $C_h = 2$
b) For circular shapes $C_h = 1.2$

C2 — 4.5.2.3 **Vertical Load**

The lift induced vertical load is not uniformly distributed across the width of the deck. The requirement to apply the total vertical load as an equivalent line load at the windward quarter point accounts for this eccentricity and the resulting torsional moment about the longitudinal axis. The application of this load is along a line, or a curve in the case of curved structures, through the windward quarter point of each cross-section of the deck.

For most superstructure types, the prescribed value of $C_v = 1$ leads to conservative values of both vertical loads and torsional moments about the longitudinal axis. C_v is sensitive to the angle of attack of the wind. This is defined as the angle of the wind in the vertical plane with respect to the horizontal. A higher value of C_v may be appropriate if the angle of attack is significantly changed from zero by the local topography. For example, in the case of bridges spanning a deep valley.

C2 — 4.5.2.4 Wind Load on Live Load

A minimal live load is expected to be present on the bridge during the 50-year return period wind conditions. The factors specified in Section 2 for the joint occurrence of wind loads on the structure and on the live load reflect the improbability of the full loads being present simultaneously.

C2 — 4.5.3 Design of Substructure

C2 — 4.5.3.1 General

In all cases, the loads transferred from the superstructure are taken to act simultaneously with those acting directly on the substructure. As noted in C2 — 4.5.2.1, for some structural systems the loads on the substructure must be taken into account in the design of the superstructure.

C2 — 4.5.3.2 Wind Loads Transmitted from the Superstructure

The modification coefficients for arriving at the transverse and longitudinal components of the horizontal load transferred from the superstructure, given in Table 2 — 4.5.3.2, are based on AASHTO [9]. The coefficients given for the horizontal transverse component also apply for the vertical load prescribed in Clause 2 — 4.5.2.3.

Both vertical and horizontal loads are applied as equivalent line loads. The latter are applied at the centroid of the exposed frontal area at each section of the superstructure. For a horizontal deck of constant depth, and with solid barrier walls and railings, the horizontal loads are applied along a horizontal line at the mid-height of the frontal area.

For normal girder and slab type highway bridges with spans of 50 m or less, Table C2 — 4.5.3.2, taken from ASSHTO [9], provides a conservative combination of prescribed horizontal loads.

Table Conservative Horizontal Load Combinations C2 — 4.5.3.2

Type of Horizontal Load	Percentage of Horizontal Load Applied	
	In Transverse Direction	In Longitudinal Direction
Forces transmitted from superstructure Clause 2 — 4.5.2.2	100	25
Forces on Live Load, Clause 2 — 4.5.2.4	100	40

C2 — 4.5.3.3 Loads Applied Directly to Substructure

Appropriate values of C_h for circular piers and columns for which $D\sqrt{qC_e} > 5.2$, where the diameter D is in metres and q is in Pa, are as follows:

- Normal steel or concrete surface: $C_h = 0.7$
- Rough surfaces with protruberances of around 0.02D: $C_h = 0.9$
- Very rough ribbed surfaces with rib heights of 0.08D and higher: $C_h = 1.2$
- A value of $C_h = 1.2$ is appropriate for circular shapes with $D\sqrt{qCe} \leqslant 5.2$
- $C_h = 2$, prescribed for rectangular and square shapes, is a conservative value intended for sharp-edged supporting elements.

The values of C_h for shapes with well-rounded corners may be reduced if justified by representative experimental data.

C2 — 4.5.4 Luminaire, Sign and Traffic Signal Supports, Barriers and Slender Structural Elements

C2 — 4.5.4.1 General

Only drag and vortex shedding induced loads are considered to be important for most support structures and structural elements. Nevertheless, for some exceptionally slender and flexible prismatic elements such as ties and iced cables, galloping excitation due to negative aerodynamic damping may be important. This excitation mechanism is described in Reference 45.

C2 — 4.5.4.2 Horizontal Drag Load

C2 — 4.5.4.2.1

Horizontal drag coefficients presented in Table 2 — 4.5.4.2 are adapted with some modification from AASHTO [40]. As indicated, the drag coefficient for rounded shapes depends on the Reynolds number. The onset of critical flow conditions, leading to reduced values of C_h, for cylindrical shapes occurs for $D\sqrt{qC_e} = 3.6$. This corresponds to a Reynolds number of 3×10^5.

The values of C_h provided for truss type supports are most applicable for solidity ratios (defined as the ratio of exposed frontal area to gross frontal area) from approximately 0.2 to 0.3. For higher solidity ratios these values are conservative. For solidity ratios of less than 0.2, use should be made of experimental data [48] or, alternatively, the overall drag should be taken as the sum of the loads on individual members. An allowance for shielding, as indicated in Table C2—4.5.2.2, is appropriate.

Horizontal drag coefficients for members, sign panels, barriers, and other shapes not included in Table 2—4.5.4.2 may, in accordance with Clause 2—4.5.1.7, be established from representative wind tunnel tests in which comparative tests are made on similar shapes included in this table. For free-swinging traffic signals, wind loads calculated using the drag coefficient given in Table 4.5.4.2.1 may be modified on the basis of experimental data.

C2—4.5.4.3 Assessment of Wind Loads Acting on Various Components

The provisions of this clause have been adapted from AASHTO [40]. The wind loads W_h, W_ℓ, W_p, and W_v for components of the structure are obtained by multiplying F_h calculated in Clause 2—4.5.2.2 by the exposed frontal area of the respective component. The ice accretion load to be used in combination with that of wind is prescribed in Clause 2—4.6.

C2—4.5.4.4 Across-Wind Loads

See Commentary on Appendix A2—2

Ice Accretion Loads on Exposed Surfaces

C2—4.6

Background information on the accretion of ice on exposed surfaces is provided in the Commentary on the Climatic Data given in Appendix A2—1. The design ice thicknesses, given in Appendix A2—1, are provided for exposed objects which obstruct the air flow. In such cases greater ice accretion can occur during periods with significant wind speed. An allowance for the action of wind has been included in the data.

For objects of relatively compact cross-section, ice accretion can occur around the entire perimeter. The design ice thickness should therefore be applied around all surfaces of all exposed objects with the exception of plate like shapes such as sign panels, bridge girders or solid barriers, where ice accretion is likely to occur only on one of the sides. Although the actual ice thickness is likely to vary around the perimeter of an exposed object, such a degree of refinement is difficult to include without further information. The use of a uniformly distributed ice thickness is considered to be adequate for most situations.

The unit weight of the accreted ice layer is taken to be somewhat greater than that of pure ice in order to allow for possible impurities.

Loads Due to Ice Pressure

C2—4.7

C2—4.7.1 and C2—4.7.2 General

Dynamic Ice Pressure

Clauses 2—4.7.1 and 2—4.7.2 are based on the provisions of CSA Standard S6-1974 [10]. Three changes have been introduced in Clause 2—4.7.2, as follows:

i) An aspect ratio coefficient C_A has been added to the expression for F, as proposed by Neill [49]. Since the values of p given in CSA S6 [10] were intended for use without an aspect ratio coefficient and are applicable for a typical massive pier ratio of w/t = 2, for which the coefficient C_A is 1.87, the unit pressures have been adjusted in the ratio 1:1.87.

ii) Tabulated values of C_N given in CSA S6 [10] comprise a step function which invites the use of the lower limits of nose angle for the values of C_N given. In Clause 2—4.7.2, graphic representation is substituted in the form of a smooth function which passes through the mid values of the ranges given in CSA S6 [10], but permits an appreciably lower value of C_N for a nose inclination of 45°.

iii) When the angle of inclination of the pier nose from the vertical exceeds 30°, lifting and flexural failure of ice sheets results, rather than crushing. When lifting occurs, the dependence of force on pier width lessens [49]. The use of 2t is conservative and insistence on a semi-circular or wedge-shaped nose increases the probability that an ice sheet would be broken before contact on an area larger than that assumed could occur.

C2—4.7.3 Static Ice Pressure

No prescriptive and definite recommendations can be made regarding ice pressure on piers frozen into large sheets of ice. Large pressures can be generated in such circumstances due to thermally induced dimensional changes in the ice sheet, changes in water level accompanied by the formation, filling and re-freezing of cracks, and wind and stream flow drag on the ice sheet. When bridge piers must be located in a body of water likely to form a large unbroken sheet of thick ice, it is recommended that a study of the site be carried out by specialists to determine design requirements and to investigate alternative solutions.

Earthquake Effects C2—4.8

C2—4.8.1 General

The earthquake resistant design of bridges requires the provision of adequate strength and ductility. Bridges should be designed to resist the forces specified in the clause without appreciable permanent distortion, and must also be able to maintain their resistance while undergoing substantial inelastic deformations. The forces specified are for structures that can safely deform well into the inelastic range and are substantially lower than those the structure would experience if no plastic deformation occurred. This clause follows the approach of the AASHTO [50] Specifications, but the response coefficients, C, given in Figure 2—4.8.2 have been modified to permit the use of regional data provided in the Climatic Appendix A2—1. This still results in structures with comparable levels of safety in areas of Canada and USA with similar seismic risk. Past behaviour of bridges subjected to earthquakes shows that the abutments, piers and bearing are more vulnerable than the foundations or the bridge superstructure. The performance of a bridge is likely to be influenced more by the joint restraint details, shear key details, details connecting the columns to the superstructure and the confinement of columns than by the design force levels. Weak or brittle connections and details may fail before the load-resisting capability of the structure has been developed.

C2—4.8.2 Equivalent Static Load Method

It is convenient to formulate the equivalent load in terms of the weight of the structure rather than the mass since the weight is used elsewhere in the design. The weight used, W, is the weight of that part of the structure which is engaged in the vibratory motion. Applying an equivalent force at the centre of mass is a simplification satisfactory in most cases. In cases where the strength or stiffness of the supports varies considerably, or the superstructure cannot be regarded as a rigid body, a distribution of the force in accordance with the distribution of mass is necessary. In Figures 2—4.8.2(a), (b), (c) and (d), the effects of soil amplification, structure ductility, structural damping ratio and foundation factors have been taken into account and further reductions should not be made.

Requirements for Seismic Zone 0 are to ensure that all structures have some minimum nominal capacity to resist horizontal forces.

C2—4.8.3 Response Spectrum Method

Since the combined response curves for C given in Figures 2—4.8.2(a), (b), (c) and (d), take into account the effects of soil amplification, structure ductility, structural damping ratio and foundation factors, they should be used as design response spectra without further reducton.

An example of the application of the Response Spectrum Method is given in Commentary K of the NBC [51].

C2—4.8.4 Special Cases

In such cases, guidance should be sought in current literature and from qualified specialists.

C2—4.8.5 Design of Restraining Features

A simple span, fixed at one end and free to slide longitudinally at the other, will have W_c equal to the entire superstructure dead load weight for longitudinal forces at the fixed abutment, and W_c equal to half that amount for transverse forces at each abutment. For restraining features at bearings which separate two parts of a superstructure, W_c should be the weight of the lighter of the two parts, and the shears due to Q in the columns supporting the lighter part should be subtracted from 0.25 W_c.

C2—4.8.6 Bearings and Supports

In an earthquake, a bridge may experience displacements four to six times those determined by elastic analysis from the forces specified in Clauses 2—4.8.2, 2—4.8.3, and 2—4.8.4. The forces prescribed presuppose this deformation capability. Therefore, to prevent premature failure, the bearings and supports must be designed in such a manner that the structure can undergo displacements of six times the movements computed from the elastic analysis.

C2 — 4.8.7 Detailing

It is clear that the Special Provisions for Seismic Design of CSA Standard A23.3M apply to structures in Seismic Zone 3, and that normal detailing requirements are adequate for structures in Seismic Zones 0 and 1. This Code takes a conservative position in requiring the application of the Special Provisions to structures in Seismic Zone 2.

In reinforced and prestressed concrete bridges, confinement and shear steel should be designed and detailed in such a manner that the full flexural resistance of the members can be developed before premature shear or anchorage failures occur. Design of shear reinforcement, column confinement steel and detailing procedures should be based on the Special Provisions for Seismic Design of CSA Standard A23.3M.

The vertical component of ground motions causes a relatively small fluctuation in effective dead loads. For unusual structures, this may require particular attention.

Inverted pendulum structures, consisting of a heavy superstructure restrained in the horizontal plane only by slender columns, need special care in design and detailing of shear and confinement steel for columns and connections between the columns and the superstructure and foundations.

Loads Due to Stream Current C2 — 4.9

C2 — 4.9.1 Load Along Longitudinal Axis of Pier

C2 — 4.9.2 Lateral Load

The formulae given are adapted from CSA Standard S6 [10]. The more favourable drag coefficients from that specification for wedge type pier nose angles of less than 90° are not given because such pier noses are more prone to catching drift. If significant quantities of drift are likely to collect at a pier nose, the area, A, for the drift should be estimated, the drift drag force, P, computed and applied at an appropriate estimated level, which will generally be higher than that of the drag force on the pier itself.

The angle referred to in Clause 2 — 4.9.2 and the directions of P_p and P are shown in Figure C2 — 4.9.2.

Collision Load C2 — 4.10

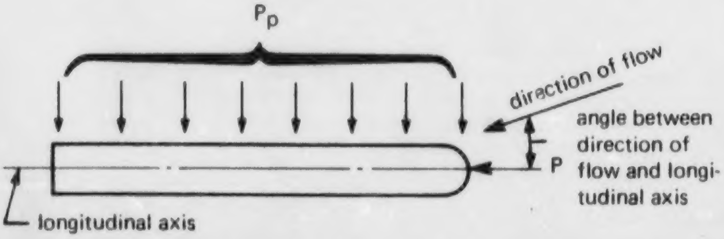
Barriers in current use may not be able to stop a heavy commercial vehicle from hitting a vital bridge pier. Failure of the pier may cause a bridge to collapse with serious consequences. To prevent collapse, highway bridge piers near the edge of the road surface must be designed to withstand the maximum dynamic force due to such collisions. If the distance between the edge of the road and the pier is 10 m or more, the probability of a severe collision is considered to be negligible.

Pedestrian bridge piers need not be designed for the collision load, since the consequences of collapse are less serious. Such piers should be protected by vehicular barriers.

The numerical value of the specified collision load is based on the loads used in the West German Standard DIN 1072 [52]. It was verified considering the load and performance factors and maximum lateral impact force due to a 450 kN vehicle on a traffic railing, as given in Table C2 — 4.3.5.2.

Construction Loads and Loads on Temporary Structures C2 — 4.11

Figure **Plan View of Pier Showing Direction of Forces** **C2 — 4.9.2**



C2—4.11.1 **General**

Earthquake is not normally considered. Although a 10-year return period for wind loads is excessive in many cases, the reference wind pressures for that return period are conveniently listed in Appendix A2—1 and the reference wind pressures for shorter return periods would not be much less. If fixed construction schedules justify neglecting or reducing seasonal loads, the assumptions made should be indicated on the drawings.

C2—4.11.2 **Dead Loads**

Bulk materials may still be considered as temporary dead load, although subject to movement, if the addition and the removal of the load are accomplished in small increments.

C2—4.11.3 **Live Loads**

Appropriate allowances should be made for dynamic effects in estimating live loads. The weight of structural components will in some cases be considered as live load while being moved into position and thereafter as dead load.

C2—4.11.4 **Falsework**

It is not intended that loads from the Code should be used in designing to other specifications.

Load Factors and Loading Combinations C2—5

General C2—5.1

The objective of the Code is to ensure a prescribed uniform level of reliability for all bridges within the jurisdictional area of this Code. For various limit states, the load factors are determined on the basis of an acceptable probability of the factored loads being exceeded during a specified time period. The limit states considered in this Code are divided into two categories:

1/ Ultimate Limit States; these include the limit states pertaining to structural safety.

2/ Serviceability Limit States; these include the limit states which may restrict the intended use of a bridge.

Fatigue as a consequence of repeated loading is a safety consideration, strictly speaking. However, the fatigue load level and its frequency of occurrence make it convenient to treat fatigue formally as a serviceability limit state. The serviceability limit states are further subdivided into two types. Serviceability limit state Type I includes structural fatigue and the human response to bridge vibration. Serviceability limit state Type II pertains to cracking and permanent deformations.

For ultimate limit states, the maximum effect of each specified load multiplied by its load factor has a small probability, in the order of 1%, of being exceeded in the bridge lifetime of 50 years. For serviceability limit states Type I, the factored live load is expected to be exceeded an average of once every five minutes on a Class A highway. For serviceability limit states Type II, the maximum effect of each specified load multiplied by its load factor has a probability in the order of 10% to 20% of being exceeded within one week so far as cracking in prestressed concrete components is concerned, and within 10 years for inelastic deformation in structural steel components. The higher probability of the serviceability limit states is acceptable because the consequences of exceeding a serviceability limit state are less severe.

Calibration of load factors was done for dead, live and dynamic loads and prestressing forces. For other loads and their combinations, the available data are insufficient for statistical calibration. The remaining load factors are determined to provide approximately similar structural safety as for the loads used in the calibration.

Table 2—5.1(a) is self-explanatory and follows traditional lines in many respects. Most other codes make use of a combination factor, by which load factors for each load combination are multiplied. A similar approach was used initially for the OHBDC but the load factors now shown have been multiplied by combination factors where appropriate.

Within the Code, the word "loads" has the very general meaning given in the definitions, and includes movements. In calculating movements for the design of bearings or expansion joints and in calculating effects due to the restraint of movements, the load factors are applied to the movements. Effects of factored movements are not to be factored, since double factoring is never required.

Older codes listed effects such as shrinkage and creep separately, but then generally prescribed the same load factor. The grouping of such loads under the load identifier, K, somewhat simplifies the process of identifying combined critical responses.

The load factor for K is also prescribed for the horizontal forces transmitted by sliding bearings when such forces have the upper bound value determined by limiting friction. This approach is adopted for simplicity, since it would otherwise be necessary to consider various load combinations capable of inducing slip. Unfactored vertical loads are used in calculating friction since to do otherwise would involve double factoring. For horizontal forces not large enough to cause sliding bearings to slip, for forces transmitted by bearing stops designed to limit travel, and for forces arising from the shear stiffness of elastomeric bearings, the load factor or factors prescribed for the loads tending to cause movement apply.

Total Factored Load Effect **C2 — 5.2**

C2 — 5.2.1 Attention is drawn to the definition of "factored load effect". In principle, it is the load that is factored rather than the effect; but in most cases, responses are linear and it is convenient to factor the calculated effects.

C2 — 5.2.2 Most modern methods of analysis consider elastic distortion as an integral part of the solution and it would be difficult to factor such effects in any way other than that prescribed.

Permanent Loads **C2 — 5.3**

C2 — 5.3.1 It can be critical to safety to consider stages in which assumed dead loads are not yet present or are not present over the whole structure. For example, near points of contraflexure, temporary conditions may produce bending moments opposite in sign to those finally occurring. For a section designed only for the final loading, this could be catastrophic, particularly if flanges designed only for tension and having low buckling resistance are subjected to compression.

C2 — 5.3.2 Minimum values of dead load factors need rarely be considered. Provided normal standards of construction and construction supervision apply, it may be assumed that variations from specified dimensions and assumed weights are small and random, or tend consistently towards larger than nominal values. Minimum values of the load factors are given for use in those generally exceptional cases where minimum values of permanent load effects can prove critical and where there is a significant probability that consistently low loads will occur.

Although the use of minimum dead load factor values is seldom justified, values of unity should be used whenever they yield results more critical than the maximum values, except as specified in Clause 2 — 5.3.3. For example, in the design of piers or footings, if dead load stabilizes the footing or reduces tension in reinforcement due to longitudinal forces such as earthquake or live load, a load factor of unity for the dead load should be considered together with the maximum load factors for the horizontal forces. Except as specified in Clause 2 — 5.3.4, it is not necessary to consider load factors having different values in different spans.

C2 — 5.3.3 The provisions of Section 6 are such that for the design of conventional retaining walls and abutments, where the wall stability is provided by the weight of the soil behind the wall and active earth pressures are assumed, soil pressure and dead load factors should be the same.

C2 — 5.3.4 Since the variation of segment weights within the limits specified may not be random, the correction of any cumulative imbalance is required.

Transitory Loads **C2 — 5.4**

C2 — 5.4.2 The variability for controlled vehicles is much less than for vehicles generally and a lower load factor is justified.

C2 — 5.4.3 For posted bridges, over a five-year period, there is a lower probability of extreme live loads occurring.

Exceptional Loads **C2 — 5.5** These loads rarely occur and the possibility of more than one occurring simultaneously can be neglected. Simultaneous occurrence of one exceptional load and one or more transitory loads is probable but a consideration of the types of structures involved, the parts of the structures significantly affected by the various loads and the relative magnitudes of the effects, suggests that it is not necessary to consider combinations other than that of ice accretion and wind load on structures covered by Clause 2 — 4.5.4.

Limit States Criteria **C2—6**

Ultimate Limit States **C2—6.1**

C2—6.1.1

This statement can be expressed mathematically as:

$\phi Rn \geq$ Largest S_j

where:
ϕ is the performance factor,
Rn is the unfactored resistance, and
S_j is the total factored load effect.

For structures that behave linearly,

$S_j = \Sigma\alpha_{ij}$ times the effect of the ith load

where α_{ij} is the load factor for load i in loading case j.

C2—6.1.2

There is some concern over structures that may be evaluated, and construction stages that may be investigated, for which all major loads are subject to load factors of 1.15 or less. Although this is theoretically justifiable, it is perhaps possible that some such structures may have an insufficient reserve of strength to cover loads that are not considered, e.g., strain induced by force fitting components, locked-in stresses, transverse temperature differential, hidden or unobserved minor defects or small unauthorised construction loads. This clause may be waived if approval is obtained, which would normally require a detailed review of the structure and the analysis.

Serviceability Limit States **C2—6.2**

C2—6.2.1 **General**

No general formulation is possible for serviceability limit state requirements.

C2—6.2.2 **Superstructure Vibration**

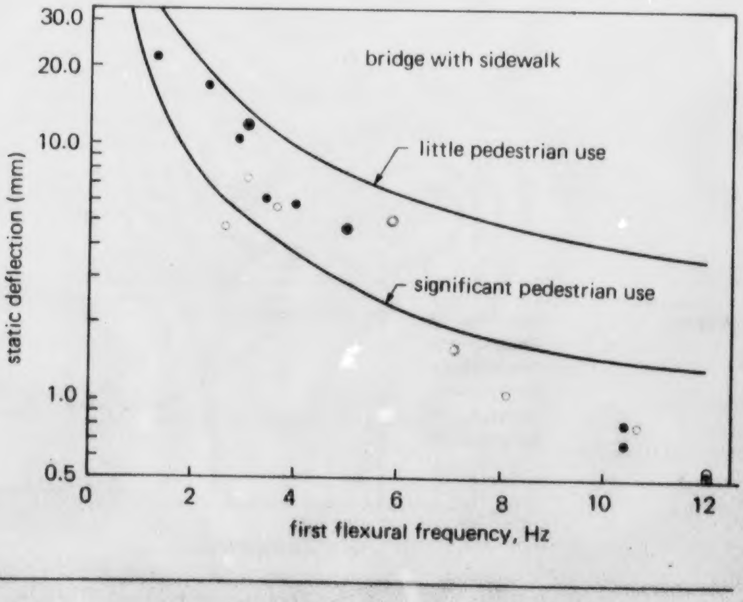
C2—6.2.2.1 **General**

The acceleration due to a representative load traversing the superstructure has been taken as a serviceability limit state for highway and pedestrian bridges. Limitations on the depth-to-span ratio, or on the live load deflection as a function of span, are not considered as providing for serviceability. The intent of such limitations has been to control vibration, prevent fatigue, limit stresses in secondary members, and account for dynamic loading. All these are provided for explicitly in this Code. For highway bridges, acceleration limits have been converted to equivalent static deflection limits to simplify the design process.

C2—6.2.2.2 **Highway Bridges**

Three types of pedestrian use of a highway bridge are considered for serviceability:

Figure **Comparison of Measured Deflections at Edge of Bridge, Adjusted to Design Load, with Deflection Criteria** **C2—6.2.2.2**



- very occasional pedestrian or maintenance personnel use of bridges without sidewalks;
- little pedestrian use, with pedestrians generally walking, on bridges with sidewalks; and
- frequent pedestrian use, generally in an urban area where pedestrians may be walking or standing on bridges with sidewalks.

The development of this clause included examination of the weight of trucks likely to cross bridges with the three different types of pedestrian use; calculation of dynamic deflection values due to these typical vehicles; and vibration limits appropriate to each of the three types of pedestrian use. A serviceability load Type I where the truck is assumed to be in a travelled lane was retained for all uses, noting that, in general, bridges with significant pedestrian use would usually be crossed by traffic at low speeds (60 km/h or less).

Values of acceleration of a typical superstructure with typical approach irregularities under the action of a typical vehicle are not easily calculated. Hence, the criteria for acceleration are expressed in terms of a calculated static deflection against first flexural frequency. The conversion of acceleration to static deflection was made on the basis of field observations of dynamic response of bridges to traffic in a travelled lane. Data from early studies [13] indicates that an average vehicle load of 150 kN results in an average dynamic deflection of 12% to 15% of the static deflection. More recent observations [14,16] for a wide range of bridge types, and vehicle loads from 100 kN to 600 kN, confirm this and show that existing bridges, whether designed with sidewalks or not, are generally satisfactory as shown in Figure C2—6.2.2.2.

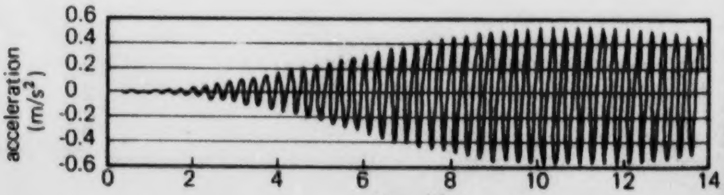
For a bridge with sidewalks, and significant pedestrian use, and where there is concern about possible traffic induced vibration, careful attention should be given in design and by quality control in construction to the minimization of bumps at the joints and undulations in the approach pavement.

Curbs and parapet walls, constructed with or without joints, can have a significant effect on the deflection at the sidewalk [14]. If a bridge satisfies the deflection criterion with the structural model used for design there is no need to consider alternatives. However, if the deflection criterion appears to govern, it will usually be more economical to recompute the deflection using an analysis which reflects bridge response under single truck loading, than to increase the stiffness of the main longitudinal members. The analysis procedures outlined in Section 3 for serviceability Type I can be used for the majority of beam-and-slab, or slab structures. Where a refined analysis is required for strength calculations, the serviceability Type I analysis can easily be included. The deflection criteria will normally be satisfied for bridges of conventional design with the longest span no greater than 20 m, or with a first flexural frequency above 6 Hz. For bridges of slab-and-girder construction, deflection may be computed at the closest girder to the specified location if the girder is within 1.5 m of that location. For bridges of slab construction, or with long cantilevers supporting the sidewalk, account should be taken of deflection due to torsion or transverse flexure resulting from an eccentric load.

C2—6.2.2.3 Pedestrian Bridges

The repeated footfall of a pedestrian crossing a flexible footbridge with little damping may result in a build up of vibration as shown in Figure C2—6.2.2.3(a). Pedestrian bridges can have less than 1% critical damping so that footfall induced vibration can result in a vibration that may be considered an almost steady-state condition. The intent of Clause 2—6.2.2.3 is to limit such vibration to a level acceptable to other

Figure **Acceleration Response of Footbridge to Pedestrian Passage, Reference 62** **C2—6.2.2.3(a)**



pedestrians. A close parallel exists between the excitation and response of floors in buildings and pedestrian bridges, so experience with human response to floor vibration as well as bridge vibration is applied here [53–61]. A selection of criteria is shown in Figure C2—6.2.2.3(b). The various criteria given recognize that the dominant mode of vibration of a continuous structure may not be the first, and the criterion chosen for Clause 2—6.2.2.3 may be restrictive in some cases. People walking alone, or in unison, have footfall frequencies of up to 3 Hz.

Pedestrian bridges with natural frequencies less than approximately 4 Hz should be given special consideration in design. If a pedestrian bridge is to be used by joggers and walkers, a detailed analysis may be warranted.

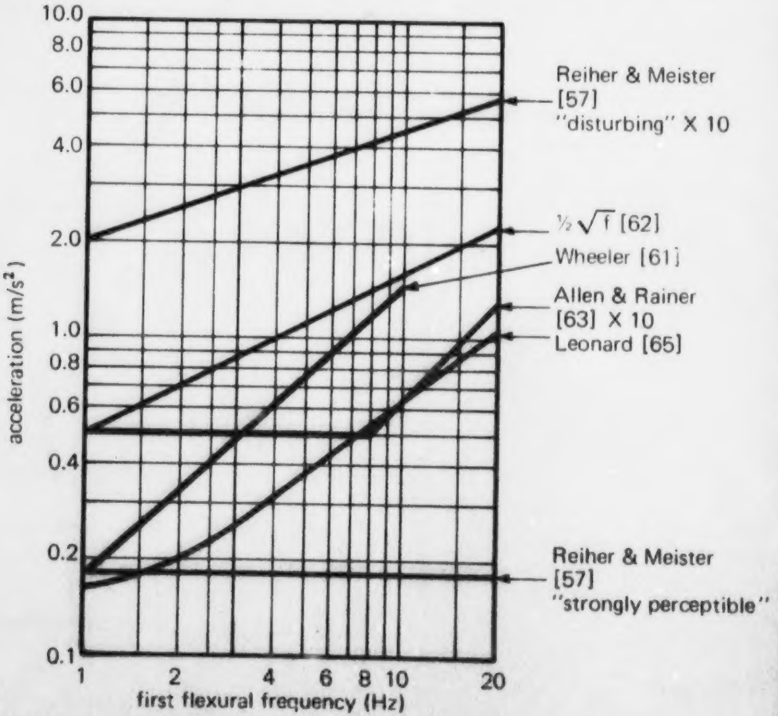
Studies of floor vibration indicate a low probability that two or more persons will walk in-phase, and a walking or standing person tends to act as a damper for vibration excited by others [53]. The same result has been found for pedestrian bridges, so the acceleration limit is based on a loading of one "typical" pedestrian weighing 700 N.

The acceleration due to the "typical" pedestrian can be calculated by a full dynamic analysis, using either the footfall impulse given in Figure C2—6.2.2.3(c) for an energetic walk [61,62] or a pulsating point load of 180 N which is approximately equivalent to the fluctuating portion of the footfall impulse [62]. The footfall frequency is equal to the lesser of the first flexural frequency of the superstructure, f_1, or 4 Hz, which is an upper limit for joggers. The pedestrian moves at a speed of the lesser of 2.5 m/s or 0.9 f_1 m/s [62], where the latter represents a 0.9 m stride at the fundamental frequency of the bridge [64].

For the majority of pedestrian bridges, a beam model neglecting any torsional effects is appropriate. For an N-span continuous bridge, at least the first N longitudinal flexural modes should be used in any analysis. For wider bridges, or bridges having similar torsional and flexural frequencies, the significant torsional modes should be included.

If a full dynamic analysis is not performed, the following simplified procedure, applicable to one-, two-, or three-span structures which act as beams, is considered equivalent to a full dynamic analysis and may be used to calculate acceleration [62]:

Figure **Criteria for Human Response to Steady Vibration** **C2—6.2.2.3(b)**



$$a = 4\pi^2 f_1{}^2 w_s K \psi$$

where:
a = acceleration (m/s²)
f_1 = first flexural frequency (Hz)
w_s = maximum static superstructure deflection (in metres) due to a vertical concentrated force of 700 N
K = configuration factor from Table C2 — 6.2.2.3
ψ = dynamic response factor, a function of damping, from Figure C2 — 6.2.2.3.(d).

In the absence of more precise information, the following values of percentage of critical damping should be used [62]:

• Steel superstructure with asphalt paving:	0.5%
• Composite concrete-steel superstructures:	0.6%
• Prestressed or reinforced concrete superstructure:	0.8%

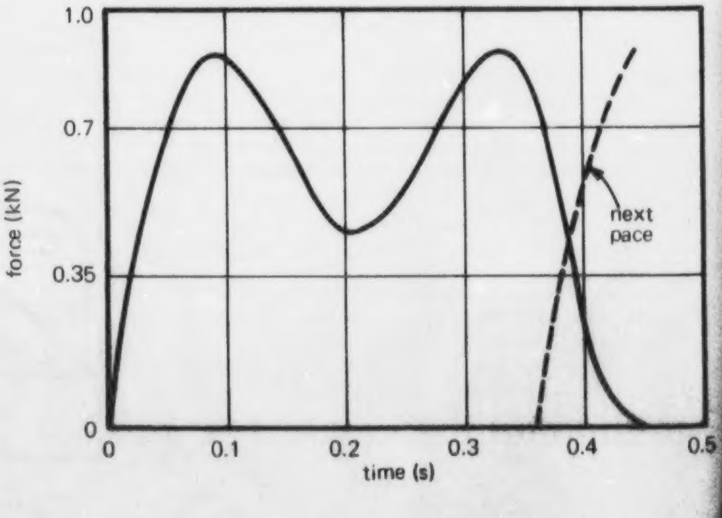
For values of f_1 greater than 4 Hz, the calculated maximum acceleration may be reduced by an amount varying from zero at 4Hz to 70% reduction at 5 Hz or greater.

If a pedestrian bridge is perceived to vibrate excessively, or when the application of Clause 2 — 6.2.2.3 results in an uneconomical structure, consideration should be given to the inclusion of devices to increase the damping of the structure [66].

Superstructures of pedestrian bridges which rest on slender piers may have rigid body modes of vibration at low frequencies. In these modes the entire superstructure may move longitudinally, transversely, or it may yaw due to flexibility of the piers. A multi-span continuous bridge may also exhibit a transverse flexural mode over the entire length of the bridge if slender supports do not provide sufficient stiffness. Slender piers should be avoided or adequately braced to avoid such modes of vibration in the frequency range of pedestrian footfalls.

To avoid coupling between the vertical mode and the transverse and longitudinal modes of vibration, a frequency separation between the vertical mode and other modes is required — hence the use of the 1.5 factor in paragraph three of the clause. Stiffness values or the mass distribution need not be adjusted for frequencies greater than 4 Hz as the footfall frequency of pedestrians seldom exceeds 2.5 Hz, about two-thirds of 4 Hz. The transverse and longitudinal modes of vibration of the superstructure will be influenced by the substructure. The restraint provided by the substructure, recognizing the flexibility of such restraint, should be included in frequency calculations for the superstructure.

Figure **Footfall Impulse** **C2 — 6.2.2.3(c)**



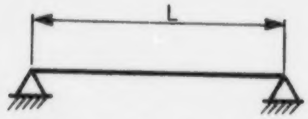
Table

Configuration Factors (K)

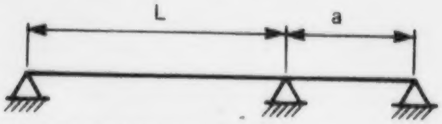
C2—6.2.2.3

Side Span Ratio a/L	2 Span Continuous	3 Span Continuous
1.0	0.70	0.60
0.8	0.92	0.82
0.6	0.96	0.92
0.4	0.96	0.92
0.2	0.95	0.90

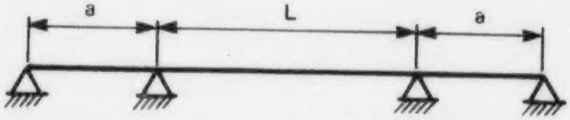
Note: (1) Configurations are shown below.
(2) K = 1.0 for simple span.



single span simply supported



2 span continuous

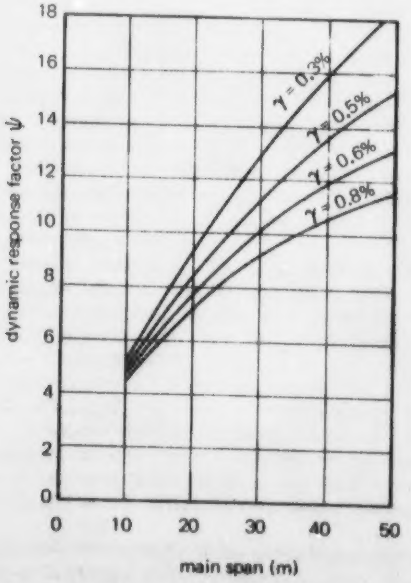


3 span continuous

igure

Dynamic Response Factor as a Function of Span Length and Damping Ratio γ

C2—6.2.2.3(d)



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Commentary on Appendix 2—1

General

CA2—1

The climatic information included in this Appendix, as well as its form of presentation, have been chosen largely to suit the specific requirements of the Code. Some information has also been added for general information. Although the Code will initially apply only to Ontario, the material is presented on a national basis since the information may be of value to other jurisdictions.

The various climatic elements presented in chart or table form are briefly described in the material below. The charts present isolines or zones of the various parameters. The tabulation presents reference design wind pressures and seismic zoning for the over 600 locations contained in Chapter 1 of *The Supplement to the National Building Code of Canada* [1]. These locations include:

- incorporated cities and towns with populations in excess of 5000 unless these are close to other larger cities;
- smaller towns and villages in sparsely populated areas; and, in some cases,
- actual weather stations in preference to locations with somewhat larger populations.

Although both the charts and the table of design values are intended to provide an overall picture of the climatic elements required for design, these may not accurately reflect the influence of the local environment. The local topography and terrain, and the presence of bodies of water or other features, may modify the local climate at the proposed bridge site. For example, winds are generally stronger in open areas, near bodies of water, and near the tops of hills and escarpments. Similarly, air temperatures and the relative humidity may be influenced by local bodies of water and local topography. All the weather data used in preparing the charts were, of necessity, recorded at inhabited locations, and hence the charts apply only to populated areas. This is particularly significant in mountainous areas where the lines on the charts apply only to the populated valleys and not to the mountain slopes and high passes, for which, in some cases, very different conditions are known to exist.

The material in this Appendix is based on data contained in *Supplement No. 1 to The National Building Code of Canada* [1] with additional information supplied by the Atmospheric Environment Service, Department of the Environment Canada, and the Boundary Layer Wind Tunnel Laboratory of the University of Western Ontario.

Maximum and Minimum Mean Daily Air Temperatures

Figures A2—1.1 and A2—1.2 present isotherms of recorded maximum and minimum mean daily air temperatures measured at a height of 1.2 m above level grassy terrain and, where praticable, away from the sheltering influence of trees and buildings. The mean temperature on a particular day has been taken as the average of the highest and lowest temperatures recorded on that day.

In the drawings of the isotherms, emphasis has been placed on data from stations with record length of 30 years and greater. The isotherms have been drawn as smooth curves to indicate overall climatic trends and thus do not necessarily reflect local departures. Detailed data may be found in *Canadian Normals Volume 1 — SI* [2] and other Environment Canada publications.

Relative Humidity

Figure A2—1.3 presents isolines of annual average relative humidity required for estimating the shrinkage and creep losses for prestressed concrete members as specified in Section 9. The relative humidity is defined as the vapour pressure of the air expressed as a percentage of the saturation vapour pressure of the air at the same temperature. The annual average relative humidity is the arithmetic average of the monthly average relative humidities for the decade of 1957 to 1966, inclusive.

The isolines of relative humidity presented in Figure A2—1.3 have been drawn as smooth curves to indicate overall trends. These do not take into account departures from these trends due to local topography, bodies of water, and other features. It is noteworthy that the relative humidity at a particular location generally exhibits a marked seasonal variation with a superimposed diurnal variation of similar magnitude. Information on seasonal variations of relative humidity is contained in *Canadian Normals Volume 4* [3].

Ice Accretion on Exposed Surfaces

Ice accretion on exposed surfaces is a common winter phenomenon. There are three main types of ice-forming mechanisms. The most important of these is the formation of glaze or clear ice during periods of freezing rain. During rainfall at or just below freezing temperatures, raindrops freeze when wetting the surface and form a high-density clear ice. Ice thicknesses on exposed objects which disturb the airflow, can be significantly increased if freezing rain occurs during periods with significant wind speed. The other two less important mechanisms are riming or cloud icing, which occurs during freezing drizzle or fog conditions at or below freezing, and hoar frost, which is formed when water vapour comes in contact with surfaces which are at below-freezing temperatures.

Ice accretion thicknesses on exposed surfaces, to be used for design, are given in Figure A2 — 1.4. These values include an allowance for the action of wind which tends to increase the accumulation of ice on exposed objects which obstruct the airflow. Ice accretion on non-obstructing surfaces, such as the roadway, are generally lower than the values given in Figure A2 — 1.4. Nevertheless, exceptions to this trend can occur in relatively sheltered areas, such as mountain valleys.

The ice thickness given for each zone is the anticipated freezing precipitation amount for a return period of 20 years, averaged over the first order weather stations within the zone. Departures from these values depending on local topography and terrain for example in mountainous regions, are possible, therefore the designer should exercise caution if ice accretion becomes an important consideration. In such cases, local sources of information should be sought. Information is also available in Environment Canada publications [4, 5].

Permafrost

The lines on Figure A2 — 1.5 indicate the approximate southern limit of permafrost and the boundary between the discontinuous and continuous permafrost zones in Canada. The distribution of permafrost varies from continuous in the north to discontinuous in the south. In the continuous zone, permafrost occurs everywhere under the ground surface and is generally hundreds of feet thick. Southward, the continuous zone gives way gradually to the discontinuous zone where permafrost exists in combination with some areas of unfrozen material. The discontinuous zone is one of broad transition between continuous permafrost and ground having no permafrost. In this zone, permafrost may vary from a widespread distribution with isolated patches of unfrozen ground to predominantly thawed material containing islands of ground that remain frozen. In the southern area of this discontinuous zone, permafrost occurs as scattered patches and is only a few feet thick.

It is emphasized that the lines on this map must be considered as the approximate location of broad transition bands many miles wide. Permafrost also exists at high altitudes in the mountains of Western Canada which are a great distance south of the southern limit shown on the map. Information on the occurrence and distribution of permafrost in Canada has been compiled by the Division of Building Research, National Research Council [6, 7].

Seismic Zones

The parameter used as the basis for establishing the seismic zones is A_{100}, and is defined as the ground acceleration with an annual probability of being equalled or exceeded of 1 in 100 [8]. This map is based on the statistical computer analysis of past earthquakes throughout Canada for this century [9]. It is corroborated by the results from a larger but less reliable seismic sample dating back to 1638.

The zones and the assigned horizontal design ground acceleration ratios for each zone are shown in the table on Figure A2 — 1.6. In the Arctic Region and other parts of the Northwest Territories, there are insufficient data for a statistical study. The zone boundaries are largely based on work by Milne and Davenport [9] and have been drawn by the seismologists of the Department of Energy, Mines and Resources.

Design and Reference Wind Pressures

Table A2 — 1.7 provides design reference wind pressures associated with return periods of 10, 25, and 50 years for some 600 locations across Canada. These reference pressures are hourly mean velocity pressures at the standard anemometer height of 10 m and are computed using the mean hourly wind speed estimated for that return period and a constant air density. The relationship between the reference pressure and the corresponding mean hourly wind speed used is:

$$q = 0.05\ V^2$$

where q is in Pa and V is in km/h.

The reference pressures for the return period of 10 years have been taken directly from Chapter 1 of *The Supplement to The National Building Code of Canada* [1]. Reference pressures for return periods of 25 and 50 years were obtained from the same source using the following interpolation procedure:

$$q(25) = 0.166\, q(10) + 0.834\, q(30)$$

and

$$q(50) = 0.576\, q(30) + 0.424\, q(100)$$

where q(10), q(25), q(30), q(50) and q(100) are the reference pressures corresponding to return periods of 10, 25, 30, 50, and 100 years, respectively.

There are considerable geographic, as well as regional differences in reference pressures indicated in Table A2 — 1.7. Generally, wind speeds are higher near large bodies of water. For example, in Ontario the highest design pressures occur along the north shore of Lake Ontario around Oshawa to Cobourg. At the same time, very much lower values are quoted for extensively wooded portions of the Muskoka and Western Georgian Bay areas. In addition, local variations are to be expected depending on local topography; for example, higher design pressures are expected near hill tops and tops of escarpments. For locations not listed in Table A2 — 1.7, the highest value for adjacent tabulated locations should be used.

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Commentary on Appendix 2 — 2

General — CA2 — 2.1

When the wind blows across a slender structure or structural element, vortices are shed alternately from one side and then the other, giving rise to a fluctuating force acting at right angles to the wind direction. This organized pattern of vortices is referred to as the von Karman vortex street. A structural element may be considered slender in this context if the aspect ratio exceeds 20. For lightly-damped structures which are free to oscillate, large amplitude vibrations in the plane normal to the wind may develop when the vortex shedding is in resonance with one of the natural frequencies of vibration. Although this is generally most likely to occur for the lower modes of vibration, vortex shedding induced effects for very flexible structures may also be important for higher modes. The character of the vortex shedding forces for circular cylinders depends on the Reynolds number $Re = VD/\nu$, where ν is the kinematic viscosity. The shedding tends to be organized at subcritical and transcritical Reynolds numbers [1, 2]. In the critical range; namely for $3 \times 10^5 \leq Re \leq 3 \times 10^6$, vortex shedding tends to be irregular unless the structural motion is sufficiently large to organize the fluctuating flow around the body. This phenomenon, referred to as "locking-in," becomes important for lightly-damped structures.

Although the mechanism of vortex shedding and the character of the resulting structural response have been extensively studied, a systematic approach to the design for vortex shedding induced effects for structures in turbulent flow is emerging only now. Although nearly periodic in a smooth airstream, the shedding in turbulent boundary layer flow conditions (characteristic of natural wind) tends to become less regular, with energy distributed over a band of frequencies around n_s. The presence of turbulence effectively reduces the extent of the structure over which the vortex shedding forces remain correlated [1, 2, 3, 4]. A reduction of the aspect ratio has somewhat similar effects [3].

Vortex Shedding Excitation **CA2—2.2**

Treating vortex shedding as a sinusoidal process, as prescribed in Clause 2—4.5.4.4 and in Appendix A2—2, is an approximation leading to conservative estimates. It may be more accurate to treat the excitation as a band-limited random process and to assume that the forcing tends to become harmonic only when the motion of the structure is sufficiently large to organize the shedding of vortices [1, 2, 4, 5]. This tends to occur when the peak amplitude of the motion is of the order of 2.0% to 2.5% of the diameter, or the width of the cross-section and greater. This approach would require approval.

The prescribed approach for structures and elements of constant diameter or cross-sectional width D, given in Part (a) of Clause A2—2.2, is intentionally severe because of the history of problems experienced with such structures. Vortex shedding induced effects for structures with constant D are particularly serious if the structural damping is low. The prescribed excitation for structures and elements with a constant diameter D varies sinusoidally at a frequency $n_e = SV/D$ with a maximum value of $F_s(x)_{max} = 1/2\,\rho C_s D V^2$. Empirical values of the coefficient C_s and the Strouhal number S are given for typical cross-sections in Table A2—2.2.

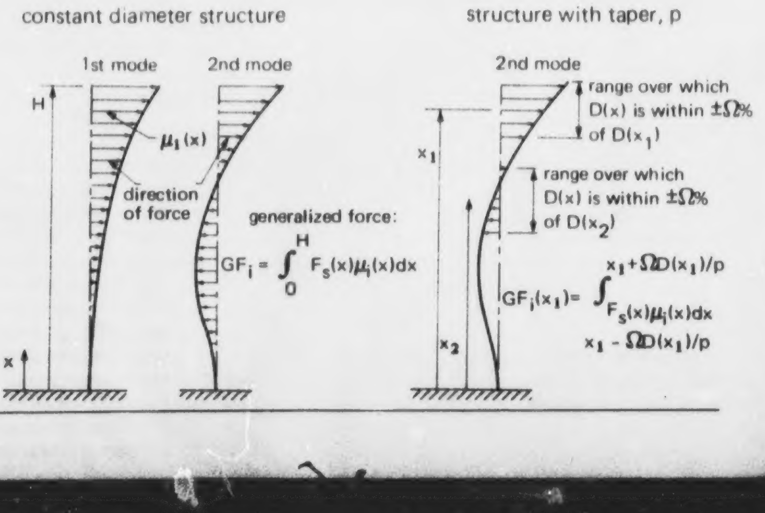
The values of C_s provided are equal to $\sqrt{2}$ times the measured RMS (root-mean-square) coefficients of the dynamic lift force. The magnitude of the excitation is taken to be invariant with x, the location along the structure, and proportional to the velocity pressure at the top of the structure or at the level of the element considered. This is a conservative assumption for vertical structures since V approaches zero at ground level. It is also conservative to treat $F_s(x, t)$ as a spatially coherent excitation, that is, acting in phase along the entire length or height of the structure. In reality, this begins to occur only at large amplitudes of motion. The variation of the wind speed with height, and the turbulent flow fluctuations normally experienced, and the presence of signs and other accessories all tend to disrupt the spatial correlation of the excitation.

The designer may wish to follow a more detailed analysis or consider representative wind tunnel model tests in cases where vortex shedding effects, computed on the above-prescribed basis, govern the design. The approach for treating vortex shedding as a random excitation with a Gaussian type power spectrum centred on n_e is discussed in References 1, 2, and 4.

The vortex shedding excitation for structures and structural elements of varying cross-section is prescribed in Part (b) of Clause A2—2.2. The prescribed excitation at a particular location $x = x_1$ is a sinusoidally-varying force at a frequency of $n_e = SV/D(x_1)$ with a maximum value of $F_s(x_1)_{max} = 1/2\,\rho C_s D(x_1) V^2$. In this case, the vortex shedding excitation is taken to remain in phase over the portion of the structure for which the diameter or frontal width remains within $\pm\,\Omega$ percent of $D(x_1)$. The value of Ω is chosen to yield estimates of vortex shedding effects by an approach based on a random shedding of vortices. Corresponding values of Ω are presented in CA2—2.3. The value of $\Omega = \pm\,10\%$, prescribed in A2—2.2, is intended as an upperbound, appropriate when the peak motion of the structure exceeds 0.02D.

For the evaluation of the vortex shedding induced response in a particular mode of vibration, Clause A2—2.2 requires that the excitation at any location be taken in the

Figure **CA2—2.2**


constant diameter structure
structure with taper, p
1st mode
2nd mode
2nd mode
H
$\mu_1(x)$
direction of force
generalized force:
$GF_i = \int_0^H F_s(x)\mu_i(x)dx$
x
range over which D(x) is within ±Ω% of D(x₁)
range over which D(x) is within ±Ω% of D(x₂)
x₁
x₂
$GF_i(x_1) = \int_{x_1 - \Omega D(x_1)/p}^{x_1 + \Omega D(x_1)/p} F_s(x)\mu_i(x)dx$

direction of the motion of the structure at that location. This is a simplifying requirement as the direction of the vortex shedding force at large body amplitudes is more likely to be in the direction of the local time derivative or velocity of the body motion. For purposes of evaluating the generalized force associated with a particular mode of vibration, both assumptions lead to the same RMS and peak values.

Typical illustations are presented in Figure CA2—2.2.

Structural Response To Vortex Shedding Excitation CA2—2.3

The evaluation of vortex shedding induced effects requires a dynamic analysis of the structure to determine its frequencies and associated mode shapes of vibration. As specified in Clause 2—4.5.4.4, all modes of vibration for which vortex shedding induced resonant vibrations occur at wind speeds equal to or less than that corresponding to the design reference pressure, namely $V \leq 1.24\sqrt{qC_e}$, must be considered. In the case of a structure with a constant cross-section, resonant vibrations for a particular mode with natural frequency N_i occur at a specific or critical wind speed, namely $V_{er} = D\, n_i/S$. In the case of a tapered structure, the frequency of vortex shedding at a particular wind speed varies over the length of the structure. As the wind speed increases, resonant excitation occurs first at the smaller diameter portion of the structure and then shifts to portions with larger diameter. Consequently, vortex shedding effects associated with a particular mode of vibration with frequency n_i must be examined for a range of critical wind speeds. Defining D_{min} and D_{max} as the minimum and maximum cross-sectional dimensions, this range is:

$$n_i\, D_{min}/S \leq V_{cr} \leq n_i\, D_{max}/S$$

Procedures for evaluating the vortex shedding induced response for various modes of vibration are provided for structures with constant and varying cross-sectional dimensions. The modal coefficient a_i corresponds to $\sqrt{2}$ times the RMS modal coefficient for the ith mode of vibration. In the evaluation of a_i for a structure with a constant D, the integration of $\mu_i\,(x)$ is over the entire length or height of the structure. The absolute value of $\mu_i(x)$ is used since the vortex shedding force at location x is in the same direction of $\mu_i(x)$.

In the case of a structure with varying D, a_i is no longer single valued but depends on the location along the structure at which the frequency of vortex shedding coincides with n_i. Consequently, $a_i(x_1)$ must be determined for all locations along the structure. The maximum value of $a_i(x_1)$ will produce the most severe force effects for the ith mode of vibration. In the case of a detailed fatigue analysis, however, contributions to fatigue damage for the entire range of $a_i(x_1)$, each associated with a different wind speed, namely $V = n_i\, D(x_1)/S$, must be considered. The integration required for the evaluation of a_i at a particular location $x = x_1$ is carried out over the part of the structure for which D(x) is within $\pm\,\Omega$ percent of D at $x = x_1$. In the case of a structure with a uniform taper p, where $p = d/d_x D(x)$, the indicated limits of integration become:

$$x_1 + b = x_1 + \Omega\, D\,(x_1)/p\text{; and}$$
$$x_1 - b = x_1 - \Omega\, D(x_1)/p$$

In the case of a non-uniformly tapered structure, the local variation of D around $x = x_1$ can be approximated by a linear taper p.

A good approximation for tapered structures is to neglect the variation of D over the limits of integration. With this assumption the modal coefficient for a support structure with a taper p for the ith mode becomes:

$$a_i(x_1) \simeq \frac{2\Omega\,\rho\, C_s\, D^4(x_1)}{p(4\pi\, S)^2\, \zeta_i\, GM_i}\,|\mu|_{ave}$$

where $|\mu|_{ave}$ is the average of the absolute values of the mode shape over the portion of the structure centered on x_1 over which D(x) is within $\pm\,\Omega$ percent of $D(x_1)$. Except near the node points, which do not contribute to maximum values, $|\mu|_{ave} \simeq |\mu(x_1)|$. The location along the structure leading to the maximum value of $a_i(x)$ for mode i can be found from:

$$\frac{d}{dx}a_i(x) = \frac{d}{dx}\,[D^4(x)\,\mu(x)] = 0$$

For a free-standing tapered support, the region of maximum excitation for the fundamental mode is at approximately 3/4 of the height and moves downward for higher modes of vibration. A value of $\Omega = \pm\,10\%$ is prescribed in A2—2. Alternative values of Ω, arrived at by matching the peak response based on the sinusoidal forcing assumption with that obtained assuming random shedding [1, 2, 4], are obtained as follows:

$$\Omega = 1.4\, g\, p\zeta_i$$

where g, the peak factor of the response in mode i due to random shedding, is:

$$g = 2\log_e(3600 n_i) + \frac{0.577}{2\log_e(3600 n_i)}$$

Values of Ω for typical ranges of p and ξ and $g = 4$ are given in Figure CA2—2.3. These values are, in some cases, somewhat higher than the value of $\Omega = \pm 5\%$ prescribed in *The National Building Code of Canada* [6] for tapered chimney stacks, observation towers, and buildings. The majority of structures covered by A2—2 fall into the subcritical Re range which leads to a somewhat longer correlation length in comparison with that of structures covered by the NBC.

The prescribed value of $\Omega = \pm 10\%$ should be used if the lower values of Ω, computed as above or taken from Figure CA2—2.3, result in a peak response of greater than 2% of D.

Although the evaluation of a_i for a particular mode must be carried out over the entire structure, the response is normally governed by the excitation from its main components. For example, in the case of lighting standards the response in particular modes of vibration is dominated by shedding from the pole, with the excitation on the luminaire bracket being of far lesser significance. As a good first approximation, the evaluation of the various modal coefficients can thus be confined to locations x_i along the pole. It is important to note that although smaller diameter components of the structure, typically luminaire brackets, may be neglected in the evaluation of the maximum value of the modal coefficient for a particular mode of vibration, such components must be included in the evaluation of subsequent force effects — in other words, the peak inertia force per unit length:

$$F_i(x) = (2\pi n_i)^2 a_i \mu_i (x) m(x)$$

must be applied for all x values or over the entire structure, including luminaires and other accessories.

Several measures may be considered should vortex shedding induced effects prove to be excessive. These include:

- a strengthening and/or stiffening of the structure;
- increasing the mass;
- increasing the damping; and
- changing the aerodynamic characteristics (for example, by increasing the taper, or by the addition of aerodynamic spoilers).

Of these alternatives, increasing the damping or the energy dissipation of the structure is the most desirable. The effective damping can be increased using visco-elastic materials or special dynamic absorbers [2].

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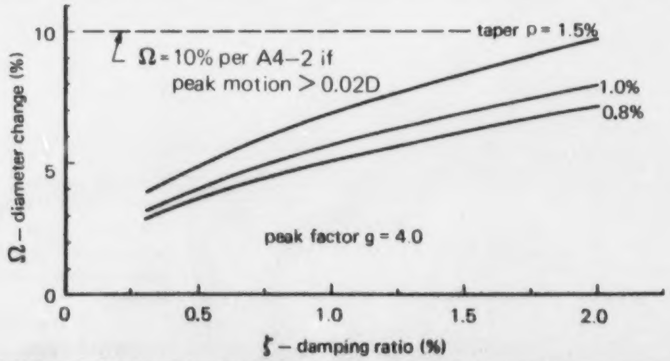
Figure **CA2—2.3**



C3 Analysis of Bridge Superstructures

Clause	Contents
[illegible]	Scope
[illegible]	Notation
[illegible]	General Requirements
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	Analysis for the [illegible]
[illegible]	Analysis [illegible]
[illegible]	Analysis of Effects [illegible]

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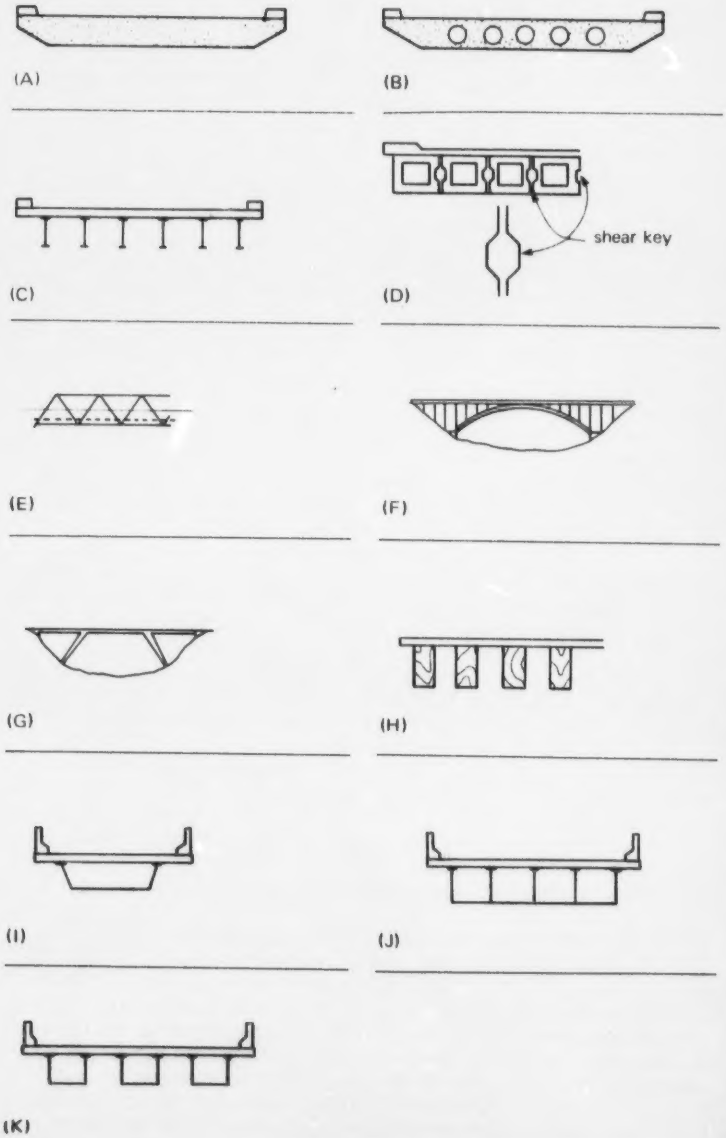
Scope **C3—1**

The bridge types concerned have been selected with a view to associating each type with one or more acceptable methods of analysis. These bridge types cover most cases of bridge construction. Figure C3—1 gives typical cross-sections of the bridge types referred to.

Notation **C3—3**

The explanations given in this Commentary involve, in some cases, symbols not defined in the section itself. These are defined in this Commentary at the places where they occur.

Figure **Representative Cross-Sections and Elevations of Bridge Types** **C3—1**



General Requirements **C3—4**

Analysis for Limit States **C3—4.1**

For many types of bridges, the pattern of load distribution at the ultimate limit state may differ from the pattern of load distribution in the elastic state. It is sometimes an advantage to the designer to be aware of this difference and to utilize non-linear methods of analysis. Since, in the present state of the art, some non-linear methods of analysis are not yet fully proven, the Code requires the use of linear analysis unless the designer secures approval for a non-linear method.

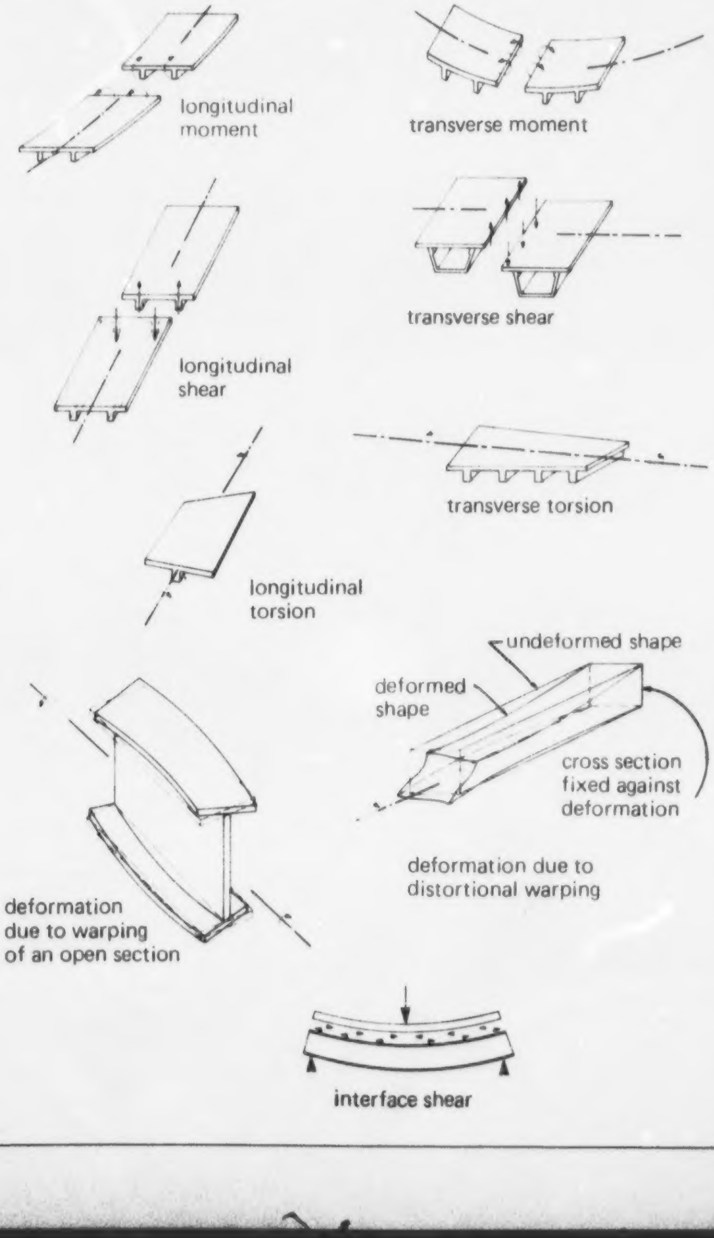
Analysis for Dynamic Loads **C3—4.2**

Methods of static analysis are frequently capable of being extended to cover dynamic analysis by the introduction of suitable dynamic allowances. This is an acceptable approach provided that it is carried out in a manner consistent with the provisions of Section 2.

Structural Responses **C3—4.3**

Certain of the structural responses listed in this clause are illustrated in Figure C3—4.3.

Figure **Illustration of Certain Structural Responses** **C3—4.3**


longitudinal moment
transverse moment
transverse shear
longitudinal shear
transverse torsion
longitudinal torsion
undeformed shape
deformed shape
cross section fixed against deformation
deformation due to distortional warping
deformation due to warping of an open section
interface shear

Factors Affecting Structural Responses **C3—4.4**

For the bridge types given in Clause 3—1, some of the structural responses given in Clause 3—4.3 always need to be considered. There are certain factors which are sometimes present in bridges of the type concerned and sometimes not. These factors when present, either give rise to structural responses not given in Clause 3—4.3 or cause changes in the relative importance of the structural responses, or both. Some of these factors are listed in Clause 3—4.4, and are described below. Appropriate analysis considerations are given in Appendix A3—1.

a) **Continuity of Spans**

The manner in which multi-span bridges may be treated similarly to single-span bridges is given in Appendix A3—1(a).

b) **Plan Geometry, including Skewness and Curvature**

Simple methods of analysis are based on the assumption of a simply-supported single-span bridge whose plan form is rectangular. When the plan form is other than rectangular, relatively large torsional effects are usually introduced; the provisions of Appendix A3—1(b) tnen apply.

c) **Edge Stiffening**

Edge stiffening introduces a local increase in the flexural rigidity per unit width of the deck slab near to the edges. Hence, a relatively large share of the total longitudinal bending moment is taken along the edges of the deck slab. Reference 40 provides details of the method given in Appendix A3—1(c).

d) **Longitudinal Variation of Transverse Cross-Section**

In the case of continuous bridges with haunches over the supports, the assumption of constant depth in the longitudinal direction equal to the depth at midspan will result in overestimation of the degree of transverse load distribution over the supports. However, the distribution of moments and shears in the middle region of this span may be taken to remain unaffected by such an assumption, provided that the haunches are limited to a short distance from the supports. Similarly, in shallow voided slab bridges, the neglecting of the solid sections over the supports could result in overestimation of the degree of transverse load distribution over the supports. In such bridges, the distribution of moments and shears in the middle region of a span may again be taken to remain unaffected by such an assumption, provided that the solid sections are limited to a short distance from the supports. The continuity of spans again also affects the total longitudinal moment and shear at a given cross-section if the transverse cross-section is not constant. Suitable methods must be used to account for this variation.

A method, applicable to the negative moment regions, for replacing the variable-moment-of-inertia girder by one of constant-moment-of-inertia is given in Appendix A3—1(d). This method is based upon a series of analyses which are yet to be reported in the literature. An alternative to the method given in Appendix A3—1(d) is reported in Reference 44. In this alternative method, the multi-span girder of varying moment of inertia is conceptually replaced by a single-span girder of constant moment of inertia, with this constant moment of inertia so chosen as to give the same deflection under a central point load.

e) **Transverse Variation of Longitudinal Cross-Section**

With the limitation on the extent of the tapered zones of the slab, given in Appendix A3—1(e), the replacement of the tapered zones by reduced width portions of full slab depth is found by rigorous analysis to be closely correct.

f) **Diaphragms and Cross Frames**

The presence of transverse diaphragms or cross frames in the superstructure of slab-on-girder bridges does not significantly affect load distribution characteristics when the governing load cases require the simultaneous loading of all the bridge lanes. In the case of eccentric loading, the diaphragms or cross frames mainly have the effect of reducing the stresses due to longitudinal moments and shears.

In box girder bridges, the transverse diaphragms or cross frames, by preventing the distortion of the transverse cross-section, can considerably reduce the stresses which would otherwise result from such distortion. The presence of adequate diaphragms or cross frames can also enable the designer to use the simplified methods of analysis, which were developed for shallow superstructures, for the analysis of box girders. In steel box girders, where the stresses due to distortion can be substantial, the provision of a sufficient number of transverse diaphragms or cross frames can significantly improve the efficiency of the structure. However, for every structure, the increase of either the number or the stiffness of the diaphragms or cross frames beyond a certain limit has little effect on further improving its distributional properties.

i) **Barrier and Parapet Walls**
If barrier and parapet walls are structurally integral with the bridge superstructure, they will have the effect of attracting load, and the bridge must be analysed for the resulting distribution of longitudinal bending effects. On the other hand, if such a wall ceases to be structurally integral with the bridge structures, for example as a result of collision by a vehicle, the distribution of longitudinal bending moment becomes changed. Hence, there is the requirement of the first paragraph of Appendix A3—1(i), which states that both cases must be considered and the more severe one used. On the other hand, for deck slab design only, the beneficial effect of barrier walls can be relied upon with confidence; hence, the inclusion of the second paragraph.
The effective positioning of the neutral axis of the barrier wall acting as an edge stiffener of the cross-section is something which cannot be closely defined; engineering judgement is needed here.

j) **Support Conditions other than Line Support**
Many of the simplified methods of analysis are based upon continuity of support all along the support lines for the full width of the bridge. Departure from this condition should be taken into account.

m) **Creep and Shrinkage**
Although creep and shrinkage frequently do not affect the ultimate load-carrying capacity of a structure significantly, they can influence the behaviour of a concrete structure in its serviceability limit states. Excessive cracking problems can occur due to stresses caused by creep and shrinkage. It is especially advisable to analyse these effects in composite construction where differential shrinkage can cause severe stresses. Similar considerations apply to the redistribution of load effects due to creep and shrinkage in segmental bridges.
The relevant portions of the Commentary on Section 2 should also be referred to.

Deformations C3—4.5

C3—4.5.1 General

In view of the approximate nature of the deformation analysis, it is not appropriate to introduce the added complication of using dynamic properties of materials. Thus, in calculating deflections for the purposes of Section 2, it suffices to use static properties.

C3—4.5.2 Vertical Deflections Due to Dead Load

Retaining the simplicity of existing methods, it is sufficiently accurate to calculate dead load deflections by ignoring the effects of transverse curvature and considering only beam type deflections due to flexure in the vertical longitudinal plane. With a similar simplification in mind, it is also permissible, for example, to assume that the total dead load of a slab-on-girder bridge is divided equally between the girders and to calculate the deflections of the girders accordingly, provided that the girders all have equal flexural rigidity and are equally spaced, and that the transverse cross-section of the bridge is symmetrical with respect to the longitudinal centreline. Similar simplifications apply to other types of bridge, based upon a uniform transverse spread of dead load.

C3—4.5.3 Vertical Deflections Due to Live Load

a) In calculating the average deflections due to live load, it is permissible to use the same basic approach as outlined above in C3—4.5.2 and to treat the entire bridge as a beam undergoing flexure in the vertical longitudinal plane only.

b) In calculating the maximum deflection due to live load, it is necessary to take into account transverse distribution effects. The simplified method given in this clause is the same as that given in Clause 3—7.1.2.2 for the calculation of longitudinal moments. A strip of width S is identified; a load fraction S/D_d is calculated; this load fraction is applied to a defined live load; the strip is treated as a beam under the action of this load, and its deflection due to flexure is calculated. The treatment of multi-spine box girder bridges of steel-concrete composite construction has the same general formulation—a load fraction is obtained from Clause 3—7.1.5.2(b) and this load is applied to a single spine treated as a beam.

Diaphragms and Bracing Systems C3—4.6

Rigorous analysis shows that in practical cases two intermediate diaphragms per span will provide almost all of the benefit that is available, with very little further beneficial effect becoming available as the number of diaphragms is increased beyond this. Reference 43 may be found helpful.

Analysis for Redistribution of Force Effects C3—4.8

Among the principal causes of redistribution of force effects are cracking, creep and shrinkage. For example, long-term creep may significantly affect the negative moment at an interior support position of a continuous span bridge.

The effects of cracking are dealt within the Code mainly by the provision for serviceability limit states. Clause 3—4.8 requires that the effects of creep and shrinkage shall be examined.

It is noted that in simply-supported single spans redistribution of internal stresses can occur without any change in total force effect, such as total longitudinal shear force or bending moment, since these are statically determinate for a given loading. An example of this is the single-span, simply-supported bridge of composite construction. Here, as the section deforms because of creep, the girder portion of the composite section tends to accept a higher and higher fraction of the total load. However, Clause 3—4.8 does not require attention to be given to such cases. In other words, only cases where the statically-indeterminate force effect, such as longitudinal shear force and moments in a continuous-span bridge, is itself altered need be considered.

Analysis for Accumulation of Force Effects Due To Construction Sequence — C3—4.9

It is noted that the Preface to the Code carries the requirement that bridge components shall be proportioned for all critical load stages during the life of the structure, including construction.

Good engineering practice calls for the anticipation, by the designer, of the probable construction sequence. When the bridge is fully built, in a statically-indeterminate system, the values due to dead load of force effects such as support reactions and negative moments at interior supports will depend upon the construction sequence and on the effects of creep. In addition, the overall state of cracking in a concrete deck will depend upon the pour sequence employed, and this will affect the statically-indeterminate forces and moments. These will also be subject to further changes due to creep.

The designer is required to follow the anticipated construction sequence stage by stage. At each stage a linear elastic analysis is performed, using the structural properties appropriate to that stage and any locked-in force effects which occur at that stage are accumulated, along with those of preceding stages, in a progressive manner. Once again, the effect of creep on this progressive accumulation should be borne in mind.

Analysis for Effects of Prestress — C3—4.10

If the line of centres of gravity of the cross-sections of the beam is straight, and if, at a cross-section denoted by i, the tendon has a radius of curvature r_i and carries a tensile force V_i, then the situation is that shown in Figure C3—4.10(a). The out-of-plane load per unit length is then given by:

$$q_v = -\frac{V_i}{r_i}$$

It is noted that q_v is measured in the direction of increasing r, so that the minus sign indicates opposite direction, i.e., as is clear by inspection, the out-of-plane loading would be upwards, as seen in Figure C3—4.10(a).

If the line of centres of gravity of the cross-sections of the beam has a radius of curvature r_{si}, and if the tendon has a radius of curvature r_{vi}, then the situation is that shown in Figure C3—4.10(b). This situation can be treated as equivalent to that of Figure 3—4.10(a) provided that the radius of curvature r_i of the notional tendon is calculated from:

$$\frac{1}{r_i} = \frac{1}{r_{vi}} - \frac{1}{r_{si}}$$

Hence, the out-of-plane load per unit length is:

$$q_v = -\left[\frac{V_i}{r_{vi}} - \frac{V_i}{r_{si}}\right]$$

Using the notation of Figure C3—4.10(b), this can also be expressed as:

$$q_v = \frac{V_i}{\ell^2}\left[2y_i - y_{i-1} - y_{i+1}\right]$$

It should be particularly noted that r_{vi} and r_{si} are taken positive in the direction shown in Figure C3—4.10(b) and that q_v is taken positive in the same direction. Thus, if the line of centres of gravity has curvature in the other direction, r_{si} will be negative.

A similar approach may be taken in cases when the line of centres of gravity has a kink. The line of centres of gravity may be treated as straight provided that an equivalent kink is taken in the line of the notional tendon. This is shown in Figure C3—4.10(c). In this case, a concentrated load P_i appears at the kink. With the notation of Figure C3—4.10(c), we have:

$$P_i = \frac{V}{\ell}[2y_i - y_{i-1} - y_{i+1}]$$

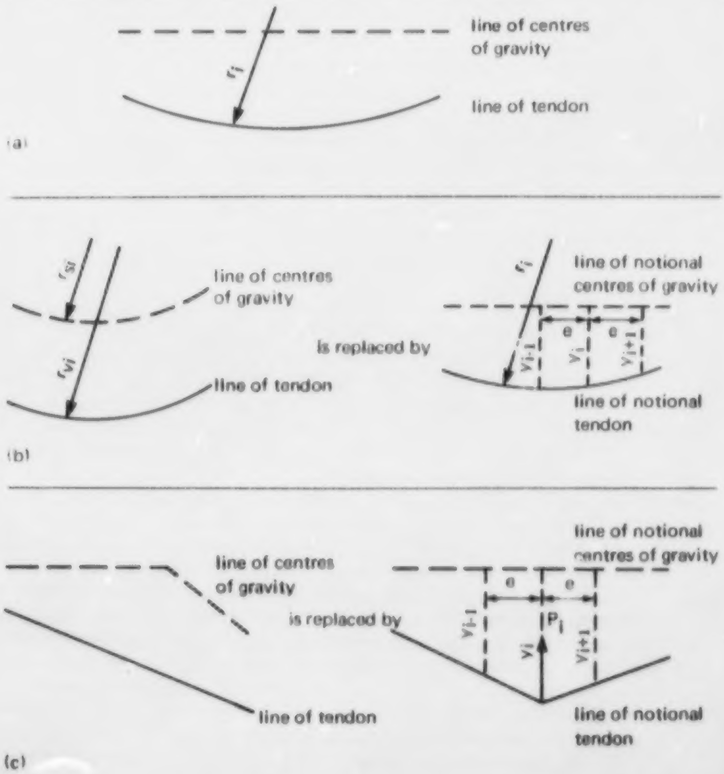
where P_i is in a direction perpendicular to the line of notional centres of gravity. An informative treatment will be found in Reference 45.

Requirements for Specific Bridge Types — C3—5

In order to be satisfactory in service, a bridge must be so designed that it can adequately resist the effects of longitudinal moment, transverse moment, longitudinal torsion, longitudinal shear, transverse shear, and in-plane forces. Methods of analysis exist which are capable of giving the effects of all of these, and such methods are quoted in Section 3. Some of these methods, for example, folded plate and finite strip methods, are such that they give all of the required information directly, without calculating explicitly the overall magnitude of any structural response such as longitudinal moment.

For certain types of bridges, it is frequently possible to use more simplified methods of analysis which do not have regard to all of the structural responses just mentioned. Such simplified methods are embodied, for example, in the AASHTO Specifications; see Reference 4. In these specifications, it is only necessary to cater directly for longitudinal moment and longitudinal shear, the remaining structural responses being looked after by the specification of minimum thicknesses of material, minimum percentages of reinforcement, and the like. In particular, for bridges with shallow superstructures, a design that is satisfactory for some of the structural responses will be found to satisfy other requirements automatically, provided that the bridges are free of complicating aspects such as heavy skew. As an example, one may consider, for a slab bridge without skew, a slab that is designed on the basis of transverse bending moment. Such a slab is invariably found to be more than adequate to accept any longitudinal torsion that arises. Again, if the slab of a slab-on-girder bridge is designed by the empirical method of Section 7, it is found that such a slab can accommodate both transverse moments and transverse shears without the need to calculate these.

Figure **Equivalent Tendon Profiles** **C3—4.10**


line of centres of gravity
r_i
line of tendon
(a)
r_{si}
line of centres of gravity
r_{vi}
line of tendon
is replaced by
r_i
line of notional centres of gravity
e
e
y_{i-1}
y_i
y_{i+1}
line of notional tendon
(b)
line of centres of gravity
line of tendon
is replaced by
line of notional centres of gravity
e
e
y_{i-1}
P_i
y_i
y_{i+1}
line of notional tendon
(c)

Slab and Voided Slab **C3 — 5.1**

C3 — 5.1.1 **Structural Responses**

It is noted that the requirement to analyse for transverse moments is retained. This is because even though, under dead load, such moments are frequently very small, there are some cases in which they must be considered. Examples are cantilevered overhang portions of decks and decks in the vicinity of isolated supports.

C3 — 5.1.2 **Limitation on Size of Voids**

In the case of voided slab bridges, there is a limit to the size of voids which can be accepted if warping of the cross-section is to be negligible. The limitations for circular voids and rectangular voids from this point of view are given in this clause. Designers should bear in mind that minimum thickness requirements for proportioning are given in Sections 8 and 9.

Truss and Arch **C3 — 5.4**

The forces in the members of a truss are calculated by linear analysis; nevertheless, stability of individual members and overall stability of the truss should be considered. The Commentary to Section 14 is recommended for reference.

Rigid Frame **C3 — 5.5**

Reference 5 should be consulted for information on the behaviour of such bridges; this work may be found helpful in complying with the requirements of this clause.

Box Girder **C3 — 5.7**

In box girder bridges, torsional warping effects are often considerable and cannot be ignored, especially in bridges of steel or steel/concrete composite construction. References 6 and 7 will be found helpful in calculating such effects.

Distortional warping is a very important aspect of behaviour of box girder bridges; its effects can be considerably reduced by the provision of adequate diaphragms or cross frames. Reference 8 provides guidance on these effects.

Dead Load **C3 — 6**

Simplified Methods of Analysis **C3 — 6.1**

In the case of dead load, the simplified method of analysis which treats the bridge superstructure as a beam is permissible provided that transverse effects are small. This means that such aspects as heavy skew and pronounced curvature-in-plan are not acceptable. Analysis as a beam is permissible, generally speaking, whenever the applied loading results in single curvature, closely, rather than double curvature.

Live Load **C3 — 7**

Simplified Methods of Analysis **C3 — 7.1**

Simplified methods of analysis for live load are based upon regarding the bridge superstructure as a rectangular plate, simply supported at its ends on unyielding line supports which are continuous across the width of the plate and do not impose moment restraint. The plate is orthotropic; that is, it has different flexural and torsional rigidities in the longitudinal and transverse directions. For such simplified methods to be applicable, the bridge superstructure must not depart significantly from the conditions of the idealized plate. The restrictions are given in Clause 3 — 7.1.1.

Shear-connected beam bridges are analysed by the methods applicable to shallow superstructures, provided that continuity of transverse flexural rigidity across the cross-section is present. If not, the analysis for longitudinal moments and shears is by the same method as for multi-spine box girders.

C3 — 7.1.2 **Longitudinal Bending Moments in Shallow Superstructures**

C3 — 7.1.2.1 **Longitudinal Bending Moments for Ultimate Limit States and Serviceability Limit States Type II**

In the case of shallow superstructures which conform to the requirements of Clause 3 — 7.1.1, longitudinal moments can be analysed by considering longitudinal strips of the structure and subjecting these strips to a fraction of one line of wheels of the OHBD truck or to the same fraction of one half of the lane load. The width of a strip is equal to the spacing of girders in slab-on-girder bridges, and is 1 m for slabs, glue-laminated wood decks, and transversely post-tensioned laminated wood decks. The fraction of the wheel loads is the ratio of the width of the strip to the value of D_d. The quantity D_d represents the total longitudinal moment at a transverse section due to one line of wheels of the OHBD truck, divided by the actual maximum intensity of longitudinal moment; it, thus, has the dimension of length.

In the AASHTO Specifications (see Reference 4), fixed values of D_d are given for various types of structure, and these values do not change with changes in cross-section or aspect ratio of the bridge. Rigorous analysis has shown that the value of D_d is not constant for all bridges of a given type. In fact, the value of D_d is determined by parameters α and θ which represent the distributional characteristics of the bridge.

Hence, in this clause, the AASHTO approach of giving a specified value of D_d is the first stage of a two-stage process, with a revised value of D_d being found after α and θ have been determined.

The method of analysis specified in Clause 3—7.1.2.1 for the ultimate limit state and serviceability limit states Type II retains much of the essential simplicity of the AASHTO approach and yet provides D_d values comparable to those obtained by refined methods. Details of the development of the method are discussed in Reference 9. It should be noted that, unlike the first edition of the OHBDC [1], the D values obtained from Figure 3—7.1.2.1 correspond to the governing load case so far as the number of loaded lanes is concerned. The modification factor for multi-lane loading, as specified in Section 2, is already implicit in the D values and therefore should not be applied further.

In Clause 3—7.1.2.1, the separate treatment of internal and external portions of the cross-section enables the designer to design the external girders of a slab-on-girder bridge differently from the internal ones. It is probable that in many cases the designer will take the smaller of the D_d values obtained for internal and external portions and use this value for the design of all girders.

The value of D_d for existing longitudinally-laminated wood bridges in which the laminates are connected only by means of nails is based upon a tire print of 0.6 m plus a spread at approximately 45° to the mid-height of the laminates.

C3—7.1.2.2 Longitudinal Bending Moments for Serviceability Limit States Type I

This clause is similar in general approach to Clause 3—7.1.2.1 and gives a method for longitudinal bending moments applicable to serviceability limit states Type I.

C3—7.1.3 Transverse Bending Moments in Shallow Superstructures

As will be seen from Reference 4, AASHTO Specifications do not require the calculation of M_y. Nevertheless, with the exception of slab-on-girder bridges which can be designed according to the empirical method of Section 7, this response can be significant and should be considered.

C3—7.1.3.1 Transverse Bending Moments in Slab Bridges

Rigorous analysis has shown that the maximum positive value of M_y is caused by only one vehicle on the bridge. The method given in Clause 3—7.1.3.1 is based on the concept described in Reference 11.

C3—7.1.3.2 Transverse Bending Moments and Associated Longitudinal Bending Moments in Slab-on-Girder Bridges

The method given here was developed by the classical small deflection plate theory for the local effects and by the grillage analogy for global effects. Details of the development of the method are given in Reference 12.

So far as local effects are concerned, the method given in this clause is also applicable to box girder bridges.

C3—7.1.3.3 Transverse Bending Moments in Transverse Wood Decking

Figure C3—7.1.3.3 illustrates the requirements of this clause, which are based upon a number of tests on existing bridge decks carried out by the Ministry and not yet reported.

C3—7.1.4 Longitudinal Shear in Shallow Superstructures

Reference 11 demonstrates that the D values for longitudinal moments, when applied to longitudinal shears, as done in some bridge codes, can lead to large errors. The characterizing parameters for longitudinal shears are not the same as those for longitudinal moments; it is noted that span length and width of bridge have little influence on transverse distribution of longitudinal shears.

The method specified in this clause was developed by a systematic grillage analysis of a large number of structures. The procedure for calculating D_d is the same as that for longitudinal moments.

Since span length is not one of the parameters influencing the distribution of longitudinal shears, the method is applicable to continuous spans as well as simple ones.

C3—7.1.5 Longitudinal Bending Moments in Box Girder Bridges

For longitudinal moments in box girder bridges to be analysed by the method of Clause 3—7.1.2, certain conditions have to be met, since this method is applicable primarily to shallow superstructures. It is essential that distortion of the cross-section, which is frequently characteristic of certain kinds of box girder bridge, be prevented if the simplified method is to apply without modification.

C3—7.1.5.1 Multi-Cell Bridges

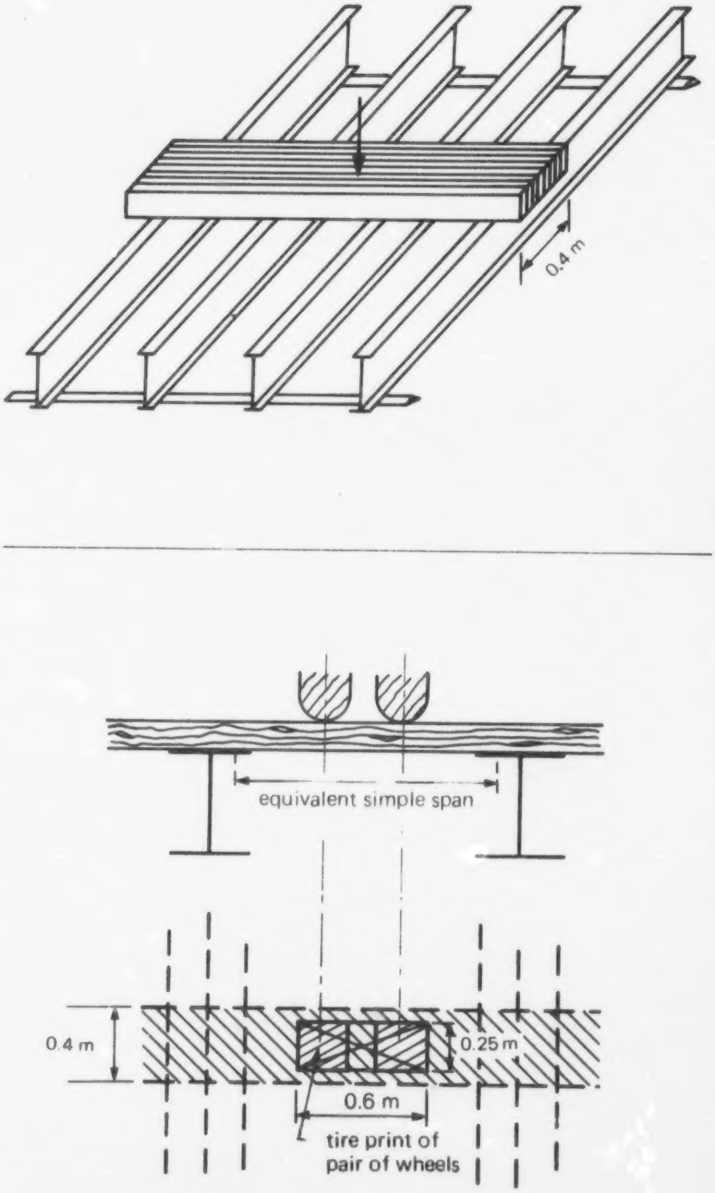
In cellular cross-section bridges, the orthotropic plate which represents the behaviour correctly is a so-called "shear weak" plate in which deflections due to transverse shear, as well as those due to longitudinal and transverse flexure and torsion, have to be considered. This additional aspect of behaviour is accounted for by means of an additional parameter δ. Once the value of δ is known, an "intensity magnifier" λ_m can be determined. The effect of this is to reduce the value of D_d in the ratio $1/\lambda_m$. Reference 10 provides details of the development of the method.

C3 — 7.1.5.2

Multi-Spine Bridges

When the transverse flexural rigidity, D_y, of a structure is very small compared to its longitudinal rigidity, the influence of the actual size of D_y on distribution properties becomes very small. Changes of D_y then result in changes to both α and θ, but the distribution properties are essentially unchanged. These distribution properties are

Figure **Load Distribution in Transverse Wood Decking** **C3 — 7.1.3.3**



then capable of being represented by the single parameter β defined in Clause 3—7.1.7. Corresponding to a particular value of β there can be many pairings of values of α and θ, all of which give the same distribution characteristics.

For multi-spine bridges, the values of D_{yx}, D_1 and D_2 are negligible compared to D_{xy}. Thus the parameter α, as defined in Clause 3—7.1.2.1(e) (ii), becomes:

$$\alpha = \frac{D_{xy}}{2[D_x D_y]^{0.5}}$$

If α is set equal to 2.0, as an arbitrary choice, then we find:

$$D_y = \frac{{D_{xy}}^2}{16D_x}$$

Substituting for D_y in the expression for θ as defined in Clause 3—7.1.2.1(e)(ii) then gives:

$$\theta = \frac{2b}{L}\left(\frac{D_x}{D_{xy}}\right)^{0.5}$$

It is noted that the choice of $\alpha = 2.0$ is indeed arbitrary. What matters is that the values of α and θ shall be consistent with one another on elimination of D_y. Rigorous analysis shows that the many combinations of α and θ that are possible give substantially the same distribution properties.

Clause 3—7.1.5.2(b) is based upon the findings of Mattock [2], which are incorporated in CAN3-S6-M78 [3]. The fraction of live load assigned to each girder has been reduced to 95% of the value previously used, by virtue of:

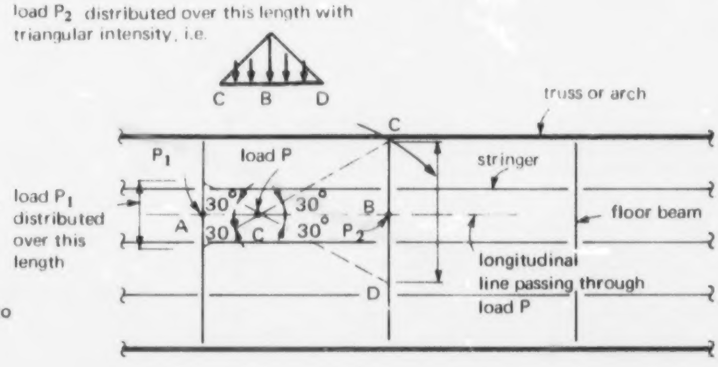
a) the presence of bracing within the spines, which leads to a reduction of longitudinal and transverse distortional warping stresses;

b) the stipulation that the cantilever overhang of the deck slab shall not exceed 1.8 m; and

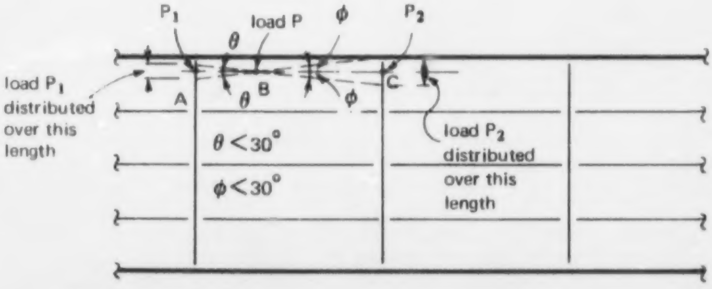
c) field observations by Holowka [41].

For serviceability limit states Type I, the torsional stiffness of girders has a beneficial effect on load distribution; for the single-lane-loaded condition, field observations indicate that a fraction of load $0.55W_L$ is appropriate [41].

Figure **Dispersion of Concentrated Load of Floor Beams** **C3—7.1.8**


load P₂ distributed over this length with triangular intensity, i.e.
C
B
D
truss or arch
C
P₁
load P
stringer
load P₁ distributed over this length
30°
30°
A
30
C
30°
B
P₂
floor beam
longitudinal line passing through load P
D

b. load distributed to less than 30°


P₁
load P
θ
ϕ
P₂
load P₁ distributed over this length
A
θ
B
ϕ
C
θ<30°
ϕ<30°
load P₂ distributed over this length

a. load distributed to full 30°

C3 — 7.1.6 **Longitudinal Shear in Box Girder Bridges**

C3 — 7.1.6.1 **Multi-Cell Bridges**

For longitudinal shear in multi-cell bridges, the "shear weak" orthotropic plate theory is again used. Calculation of the parameter δ leads to an "intensity magnifier for shear" λ_s.

C3 — 7.1.6.2 **Multi-Spine Bridges**

Clause 3 — 7.1.6.2(b) is based upon similar considerations to those outlined in the Commentary to Clause 3 — 7.1.5.2(b).

C3 — 7.1.7 **Transverse Shear in Shear-Connected Beam Bridges and Prestressed Wood Decks**

The development of the method which gave rise to Figure 3 — 7.1.7(a) will be found in Reference 42.

C3 — 7.1.8 **Analysis of Transverse Beams**

Methods employing the D value concept are not applicable to beams which span perpendicularly to the traffic direction. Traditional ways of directly apportioning loads to floor beams, without accounting for any dispersion, are slightly conservative. Provisions for distributing the concentrated loads are diagramatically shown in Figure C3 — 7.1.8. If the transverse medium is not continuous, direct supporting is applied.

C3 — 7.1.9 **Analysis of Stringers in Truss and Arch Bridges**

The panel between floor beams may be regarded similarly to a bridge between non-deflecting supports. The fact that the supports (in this case the floor beams) do, in fact, deflect improves the load distribution but the effect is marginal. Hence, the assumption of non-deflecting supports is slightly conservative.

If the stringers are continuous over the floor beams, the considerations of Appendix A3 — 1(a) may be applied.

Permitted Refined Methods of Analysis

C3 — 8

General Applications **C3 — 8.1**

Refined methods with general applications are by now well reported in the technical literature. The references given below are representative of a large number of published papers:

a) Grillage analogy — References 13, 14, 15, 16
b) Orthotropic plate methods — References 17, 18, 19
c) Finite element methods — References 20, 21, 22
d) Finite strip methods — References 23, 24, 25
e) Folded plate methods — References 26, 27, 28

Specific Applications **C3 — 8.2**

References 29 and 30 are representative of a large body of published work on influence surfaces, and may be found useful.

Model Analysis **C3 — 8.3**

In recent years, model analysis has become a well-defined specialty study. In order to be valid it must be carried out by a person or persons of wide experience, capable of designing the model or models so as to reproduce properly all relevant behaviours of the bridge and capable of interpreting the model behaviour in terms of bridge behaviour. References 31, 32, 33 and 34 give explanations of the approach.

Selection of Methods of Analysis

C3 — 9

The methods listed in Table 3 — 9 are capable of analysing the relevant structural responses for the bridge types shown.

In selecting a method or methods of analysis, Table C3 — 9 may be found useful. It indicates, in summmary form, which structural responses need to be analysed for in bridges which satisfy the conditions of Clause 3 — 7.1.1 for live load analysis. It is to be noted that Table C3 — 9 does not list "deformation" which is one of the required structural responses of Clause 3 — 4.3. Analysis for deformation should also be included.

Idealization of Structure and Interpretation of Results

C3 — 10

General **C3 — 10.1**

The requirements for idealization contained in this clause and in Appendix A3 — 2 are based upon many published papers and reports, of which References 35, 36 and 37 are representative, and may be found useful. In the particular case of wind bracing and its inter-relationship with the remainder of the superstructure, the work of Kollbrunner [39] is recommended.

Table **Structural Responses to be Calculated** **C3 — 9**

Bridge Type	Structural Responses						
	Longitudinal Moment	Transverse Moment	Longitudinal Torsion	Longitudinal Shear	Transverse Shear	In-Plane Forces	Remarks
A. Slab	√	√		√		√(1)	
B. Voided Slab	√	√		√		√(1)	Voids must satisfy size limitations of Clause 3 — 5.1.2
C. Slab-On-Girders	√(2)	√(3)		√		√(1)	
D. Shear-Connected Beams							
(a) with continuity of transverse flexural rigidity across the cross-section	√	√		√	√(4)	√(1)	The bridge shall be treated as A, B or C as appropriate, and transverse shear shall be considered for live load
(b) without continuity of transverse flexural rigidity across the cross-section	√			√	√(4)	√(1)	Longitudinal moment and longitudinal shear are calculated as for multi-spine bridges of concrete construction
E. Truss (a) Truss Members (b) Floor System	Using appropriate orientation, treat similarly to Type A, B, C, D, or H.						Axial forces for each line member of the truss shall be considered. Strength of connections, overall stability, and stability of the compression chord of a pony truss shall also be considered.
F. Arch (a) Arch Members (b) Floor System	Using appropriate orientation, treat similarly to Type A, B, C, D, or H.						Axial forces for each line member shall be considered. Strength of connections and overall stability shall also be considered
G. Rigid Frame (a) members supporting floor system (b) floor system	Using appropriate orientation, treat similarly to Type A, B, C, D, or K.						In-plane forces induced by frame action shall be considered. Axial forces, bending moment and shears for each line member shall be considered
H. Bridge Incorporating Wood Beams	√	√(5)		√	√(6)	√(7)	Live load deflections must be considered and the requirements of Section 13 followed.
I. Box Girder — Single Cell	√	√	√	√	√	√	Torsional warping needs to be considered in the case of steel and steel-composite construction only. Distortional warping need not be considered in the case of concrete or composite construction provided that: (a) adequate diaphragms or cross frames, as the case may be, are present over the supports; (b) in addition to those over the supports, a minimum of two intermediate diaphragms per span are present for concrete construction and three per span for composite construction; (c) adjacent diaphragms are spaced not more than 18 m apart in the case of concrete construction and not more than 12 m apart in the case of composite construction.
J. Box Girder — Multi-Cell	√	√	√	√	√	√	
K. Box Girder — Multi-Spine	√	√	√	√	√	√	

Notes:

1) Relates only to prestressing.
2) Taken by girder only or girder-plus-slab, depending or whether the construction is non-composite or composite, respectively.
3) Taken by slab only. Need not be considered if slab designed by the empirical method of Section 7.
4) Applies only to live load.
5) Applies only to deck-on-stringer type.
6) Applies only to prestressed wood decks under live load.
7) Applies only to prestressed wood decks.

Effective Flange Widths for Bending **C3 — 10.2**

Clause 3 — 10.2 provides the information necessary to obtain reduced values, B_e, of overhang. The total effective flange width of the cross-section, per girder or per spine, is then obtained as (left-hand overhang plus central portion plus right-hand overhang) where the central portion may be noted from Figure 3 — 10.2(a) for various types of superstructure. It is noted that the left-hand value of B_e and the right-hand value of B_e may be different, so that an asymmetrical effective section results. This will be the case when an outermost girder has an overhang B which is not equal to one-half of the clear spacing between girders.

The formula given in Clause 3 — 10.2 is an accommodation between two different curves as developed by Cheung and Chan [38]. The formula refers to effective flange width for bending.

Clause 3 — 10.2 does not deal with effective flange width for the in-plane component of prestressing force. Designers may find the following treatment helpful for the latter problem.

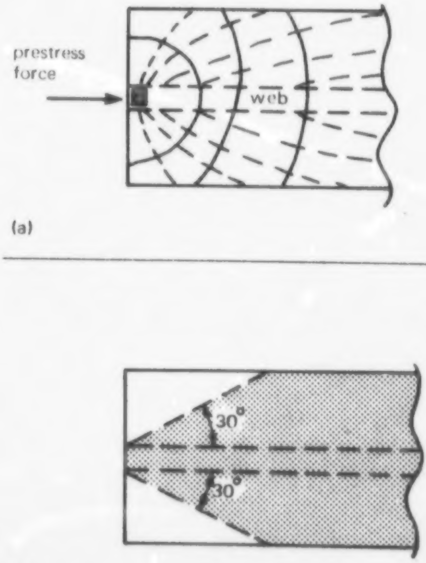
Figure C3 — 10.2(a) shows the general form of the principal stress trajectories, compressive and tensile, which arise in the deck near to an end support as a result of the prestressing force. These trajectories indicate that the effective flange width for resisting the in-plane component of prestressing force should be permitted to fall to zero as the end support position is approached, with the result that at the end support the prestressing force is resisted by the web only. The stress trajectories are also instructive in indicating the general nature of the bursting stress problem.

It is suggested that the effective flange width for prestressing force be diminished to zero in a straight line taper, as shown in Figure C3 — 10.2(b), near an end support. The angle of taper cannot be precisely defined, since the straight line is an approximation to a curved trajectory of stress, and must be selected by engineering judgement. A value of 30° is suggested.

Idealization for Analysis **C3 — 10.3**

The designer is reminded that the questions of "composite or non-composite" and "cracked and uncracked" are distinct from one another. For the purposes of this clause, the combinations to be used are "composite, with uncracked concrete" or "non-composite, with uncracked concrete," according to the presence or absence of shear connectors.

Figure **Stress Trajectories and Effective Flange Width for Prestressing Forces** **C3 — 10.2**



(a)

(b)

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C4 Deck Joints and Bearings

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C4.3 Deck Joints
C4.8 Bearings
C4.[illegible] [illegible]
[illegible] References

Scope

C4—1

Bearings and deck joints are expensive components of the structure. They are often difficult to install and replace, and they require routine maintenance. Consequently, structures should be designed to include the practical minimum number of joints and bearings. Analysis may show that for some structures, bearings and/or deck joints are not required, and they should be eliminated wherever possible.

Movements and Loads

C4—4

Bearings are normally selected on the basis of load and movement data provided by the manufacturer. These figures determine the minimum guaranteed capacity of the bearing. Similarly, deck joints are usually selected from a manufacturer's catalogue giving the movement capability of the joint. The performance factor for bearings and joints of guaranteed capacity is then 1.0. Where ultimate strength data are not available, the designer should satisfy himself that the data available at serviceability limit states will adequately satisfy all his requirements.

At ultimate limit states, the damage permissible to deck joints is limited to that which can readily be repaired, such as replacement of an elastomeric seal. Damage to the structural parts of the joint assembly (for example, the anchorages or permanent deformation of finger plates) is not permissible at the serviceability limit states. A bridge is considered to remain in service as long as traffic is maintained on the structure, even though one or more lanes may be closed to traffic to permit repairs to be carried out.

Structural movements result from either loading conditions and/or volume change. The magnitude of the movement depends upon the nature of the structure and its responses to static and dynamic forces.

The following sources of movement require particular attention.

a) Flexural deflection may contribute significantly to bearing and deck joint translations because the locations of the bearing and deck joint do not coincide with the flexural centroid of the member.

b) Tilt of the abutments may occur, even when a piled foundation is used.

c) Rotation of bearings may occur because of the imperfect alignment of bearing seats, even when the bearing seats are built to within the permissible construction tolerances. Where bearings with little or no rotation capacity are used, grinding of the bearing seats or the use of a grout bed may be required.

d) Structures translate both along and at right angles to the longitudinal axis. These translations may cause serious problems with some types of deck joints, such as finger plates. A method of analysis for computing the location of the zero movement point and the effect of the actual stiffness of individual supports in bridges is available [1, 2], both for straight structures and those which are curved and skewed. Additional information is given in Commentary Clause C2—4.4.

Deck Joints

C4—5

General Requirements

C4—5.1

Joints which are commonly considered "fixed" are usually fixed against translation, but not rotation. The rotation should be accommodated through the provision of either an open or a sealed joint.

The poor performance of deck joints has been one of the major causes of remedial work on structures. The primary cause of damage is water, often containing de-icing chemicals, passing through the deck joint. Proprietary products should be evaluated against the requirements of Clause 4—5.

The Ministry publishes a list of designated sources, DS141 *Joint Assemblies, Bridge Expansion* [3] which lists the proprietary joints meeting most of the requirements. It should be noted that none of the deck joints currently in use completely satisfy the requirements of Clause 4—5.

The movements which a deck joint must accommodate include skew movements. Some proprietary joints have been found to be permanently deformed when installed at a skew angle.

Joints installed in a substantial concrete dam have been found to be less susceptible to failures of the joint and adjacent pavement. A typical detail is illustrated in Figure C4—5.1.

Installation

C4—5.2

A 5 mm recess is the minimum requirement for protection against snowplow damage. A deeper recess is often used where the wearing course is bituminous concrete, but a recess greater than about 10 mm reduces the quality of ride significantly. Steel ribs are sometimes used to raise the plow as it passes over the joint. Additional precautions to prevent damage by snowplows are advisable when the skew of the expansion joint coincides with the skew of the snowplow blade, typically 30° to 35°.

Anchorage **C4 — 5.3**

The anchorage requirements are based on work reported by Black [4] and Lee [5]. Mechanical expanding-type anchor bolts drilled into the concrete and anchor bolts set in grout should not be used because loosening has been observed under repeated loading. Flexible bedding materials have also contributed to bolt failure. Anchor bolts installed nearer to the face of the joint than the first transverse reinforcing bar have caused spalling of the concrete at the joint.

Figure C4 — 5.1 illustrates an effective method of anchoring deck joints by reinforcing bars cast into the deck. Perforated plates have also been used satisfactorily. Wherever possible, the anchorage system should be fixed to the main structural reinforcement. In the case of perforated plates or loops of the type illustrated, this can be done by passing transverse bars through the loops.

Vent holes, typically 10 mm in diameter at 300 mm centres, should be provided in the top face of the angle to ensure that there are no voids in the concrete beneath the angle.

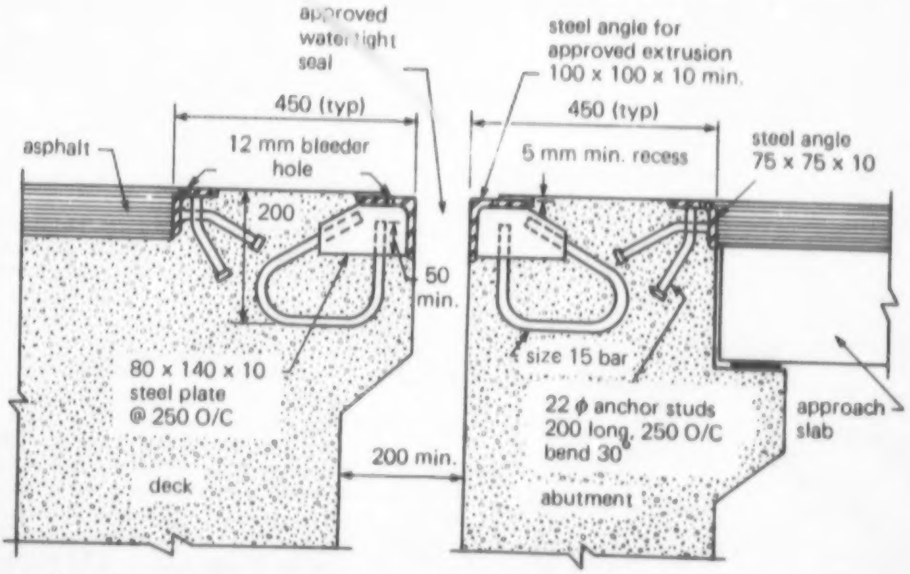
Open Deck Joints **C4 — 5.4**

Under certain conditions, an open deck joint can provide an effective and economic solution. Satisfactory performance depends upon the provision of an effective deck drainage system, control of flow of run-off through the joint, and disposal of the run-off from the site. Washout of slope paving and ponding around column bases has occurred where provision for disposal of run-off has been inadequate. Corrosion damage has occurred in the substructure components of some structures exposed to de-icing chemicals because of the lack of positive protective measures against corrosion. Similar damage has also occurred where sealed joints have leaked. It is essential that surface run-off and roadway debris not be permitted to accumulate on any part of the structure below the joint. This is usually achieved by providing pedestals sloped at 45° around the bearing seats and avoiding surfaces with a slope of less than 3 horizontal to 1 vertical.

The main factors in determining whether to use an open or a sealed joint are cost and the location of the site. The total cost of using the joint in a particular structure should be included in cost comparisons. This comprises not only the cost of the joint but, for open joints, the cost of providing for disposal of the surface run-off. The total cost of sealed joints may include the cost of providing for additional forces on the structure, and an increase in maintenance costs.

In general, open joints are well suited to secondary highways where little sand and salt is applied during winter maintenance operations. They are not suited to use in urban areas where the costs of providing for the surface run-off are high.

Figure **Typical Anchorage and Armouring Details for a Deck Joint Assembly** **C4 — 5.1**



all dimensions are in mm

Sealed Deck Joints **C4 — 5.5**

The accumulation of significant quantities of roadway debris has been observed in several proprietary-sealed deck joints. The satisfactory performance of a deck joint is dependent upon the provision for run-off from the joint and the design of the deck and roadway drainage system. Water and moist roadway debris accumulated in the joint can freeze in cold weather and restrict the movement capability of the joint. Regular inspection and maintenance is usually required to ensure satisfactory performance. Deck joints should be prefabricated in as long a section as practicable (including the curb, sidewalk, or median sections) so as to minimize the number of field splices. Field splices, where necessary, should be placed at the crown of the deck and should never coincide with the wheel path areas. Where the joint consists of an elastomeric gland inserted between metal components, it is usually possible to insert the gland in one piece even when shipping lengths require the steel sections to be field spliced.

It is desirable that all elastomeric components of the deck joint be protected from direct contact with traffic. Deck joints having elastomeric components in direct contact with the deck surface do not provide adequate support for the edges of the deck wearing surface. Deck joints which achieve a watertight seal through the use of field-applied liquid sealants rarely perform satisfactorily. Seals which rely upon an interference fit between a rubber gland and steel components have also been found unsatisfactory. A clamping device with mechanical fasteners should be used to secure rubber glands. Many proprietary deck joints exert a substantial horizontal force as the joint cycles through its movement range. This effect may become very significant on long, wide structures, and the joint anchorage, superstructure, and substructure should be designed to accommodate these forces. Where the forces are calculated from factored movements, the factors need not be applied again.

Bearings **C4 — 6**

General Requirements **C4 — 6.1**

For inclined bearings, the loads and movements required are those with respect to the axis of the bearing. Where cylindrical sliding surfaces are used, the bearing unit must be set dead level so as to ensure even pressure distribution on the bearing. The use of a spherical sliding surface is often preferable in order to allow for construction tolerances.

C4 — 6.1.2 Guidance on the design of concrete hinges is given in publications by the Cement and Concrete Association and the British Ministry of Transport [6, 7].

C4 — 6.1.4 The loads and movements to be shown on the plans are the governing cases calculated by using the factors given in Table 2 — 5.1(a).

Installation **C4 — 6.2**

The temporary connection in pre-assembled bearings with a TFE sliding surface should not be removed until after placing the bearings in the structure, in order to prevent contamination of the sliding surface. Although installation settings are shown on the plans, it is often economical to extend the top plate of the bearings so that it can be set symmetrically and without the need for adjustments at the time of installation. This practice reduces the possibility of improper settings and eliminates the risk of the bearing being installed the wrong way round. It is also good practice to mark the direction of movement on the bearing, particularly on bearings which are restrained against lateral movement. In any event, the settings must be such as to accommodate the total movements calculated in accordance with Clause 4 — 6.1.4.

The bedding grout should only be used to provide a plane surface and not to correct poor workmanship in the construction of the bearing seat. Because it is unreinforced, the thickness of the grout is limited so as to minimize the possibility of cracking. Temporary shims should be removed to prevent them from causing stress concentrations. In some circumstances, a permanent lead sheet may be preferable to a bedding grout.

Rollers, Rockers and Sliding Plates **C4 — 6.3**

Small-diameter rollers do not perform satisfactorily. Corrosion takes place in the base plate immediately below small-diameter rollers, and translation is prevented.

Roller nests consisting of several small-diameter rollers are usually unsatisfactory. The load is rarely distributed evenly between the rollers because of non-uniform deformation of the top and bottom plates. Rollers side by side are preferable to rollers in tandem. Rollers in tandem should be surmounted by a rocker arrangement so as to accommodate rotations and ensure equal distribution of the load. The requirements of Clause 4 — 6.3.4 are illustrated in Figures C4 — 6.3(a) and C4 — 6.3(b).

The coefficient of friction varies with the material and the geometry of the bearing. Typical values for new bearings are given in Table C4 — 6.3. In the evaluation of existing structures, due consideration should be given to the condition of the bearing. In many older structures, particularly those with steel bearings, the bearings are frozen. This is a factor in rating the capacity of the structure.

TFE Sliding Surfaces **C4 — 6.4**

For TFE sliding elements, the 45 MPa limit on the contact pressure at the serviceability limit states Type II is necessary in order to limit the creep of the TFE. The TFE element should not be used to accommodate rotation.

The coefficient of friction of TFE is affected by a number of factors including contact pressure, roughness of sliding plate, previous wear, temperature and sliding velocity [8]. The friction coefficients are taken from AASHTO [9]. The coefficient of friction assumed in design should always be, within the limits given, that which produces the critical condition. For example, when proportioning a fixed bearing or support, the value of the coefficient of friction used for the expansion bearings should be carefully selected to ensure that horizontal loads applied to the fixed support produce the critical condition.

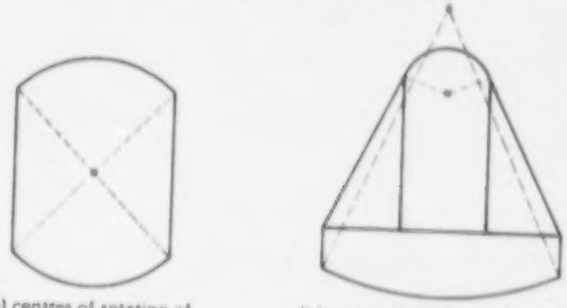
The choice of fillers such as glass fibre, graphite or bronze has a pronounced effect on the coefficient of friction. Since the fabrication techniques can have a considerable influence on the properties of the filled sheet, the choice of filler should be determined on the basis of data provided by the bearing manufacturers.

Welding to steel plate which has a bonded TFE element is possible provided that welding procedures are established which restrict the maximum temperature of the interface to 150°C.

Table **Typical Coefficients of Friction for Various Types of New Bearings** **C4 — 6.3**

Type of Bearing	Coefficient of Friction
Flat steel sliding plates	0.30
Steel sliding plates with curved top plate	0.20
Lubricated bronze sliding plates	0.10
Steel rollers	0.05

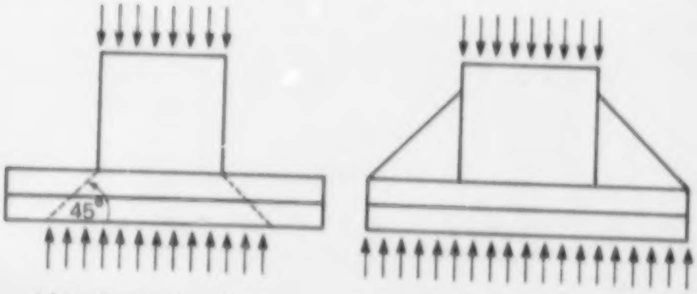
igure **Acceptable Relative Location of Centres of Rotation** **C4 — 6.3(a)**



(a) centres of rotation of upper and lower surfaces coincide

(b) centre of rotation of lower surface higher than centre of rotation of upper surface

gure **Method of Calculating the Length of Rocker Bearing Surfaces** **C4 — 6.3(b)**



(a) cantilever neglected: no stiffening

(b) cantilever included: with web stiffening

The sliding surfaces should be arranged so that sliding movements will cause accumulations of dust and dirt to fall from the mating surface. This arrangement is usually possible except in the case of long-span steel girders where the location of the web stiffener does not coincide with the bearing reaction throughout the movement range. In such cases, additional stiffening must be provided or the bearing inverted.

Unconfined Elastomeric Bearings **C4 — 6.5**

Some laminated bearings are manufactured with alternate elastomer and fabric laminae. In general, this type of unit has a much greater compressive strain for a given applied pressure than a similar steel laminated bearing. Steel laminated bearings with other than integrally moulded edge protection have been found to be unsatisfactory in service. Laminated bearings which are not moulded as a single unit under heat and pressure are susceptible to bond failure between the laminae.

The current practice in Ontario is to permit the use of natural polyisoprene (i.e., natural rubber) or polychloroprene (most commonly known by the tradename Neoprene). The most significant difference between the two polymers is that polychloroprene has superior heat and oxidation resistance, whereas polyisoprene has better compression set and shear resistance properties, especially at low temperatures. Where temperatures of less than −25°C are expected for prolonged periods, the use of only polyisoprene should be considered. Tests should verify that the elastomeric compound selected retains its flexibility under the lowest anticipated service temperature.

Geometric proportions are given in order to ensure stability of the bearings. In order to encourage standardization and permit the substitution of elastomeric bearings from different sources, the Ministry uses, wherever possible, the plan sizes given in Table C4 — 6.5.

On structures subject to irreversible movements, notably post-tensioned structures, it is often feasible to provide an elastomeric bearing to accommodate only the reversible movements. The irreversible movement can be accommodated by using the elastomeric bearing in combination with a sliding element or by jacking the structure before the bearing reaches its movement capacity and after most of the irreversible movement has taken place. Either of these methods is often more economical than providing a much thicker bearing to accommodate the total anticipated movement.

Horizontal crawling of the bearing is likely to occur wherever the combination of light loads, a significant amount of movement, and inadequate attachment of the bearing exists. The coefficient of friction between elastomer and painted steel varies from 0.10 to 0.15. The coefficient of friction between elastomer and broom-finished concrete is about 0.20, and to smooth, steel-trowelled concrete is the same as to steel.

The maximum average unit pressure of 7 MPa is not itself a limiting condition, but is specified in order to limit the deflection of the bearing. The limitation on the dead load is necesary to control the creep of the elastomeric component of the bearing.

The ultimate compressive strength of a steel laminated bearing manufactured to the requirements of Clause 4 — 6.5.1 is typically in excess of 40 MPa. Failure usually occurs in bond or by fracture of the steel plates.

The compressive strain of elastomer is determined by the average compressive stress, the hardness of the elastomer, and the shape factor. Elastomeric layers with a shape factor of about 8 will meet the requirements of Clause 4 — 6.5.1. For elastomeric layers with a shape factor greater than 12, the compressive strain is very small and can be neglected in calculating the compressive deflection of a typical bearing. For most laminated bearings, only the top and bottom layers will have a shape factor greater than 12. Bearings with a high shape factor are undesirable because their rotational capacity is limited by the small compressive strain which occurs in such bearings.

Table **Standard Bearing Plan Sizes (mm × mm)** **C4 — 6.5**

300 × 200	350 × 250
400 × 300	450 × 350
500 × 400	550 × 450
600 × 500	600 × 600

Overall height should be a multiple of 10 mm.

Figure 4 — 6.5.3.5 has been constructed for the following criteria.

a) There is no rotation.
b) The deflection at serviceability limit states Type II does not exceed $0.07T_e$
c) The average pressure at the serviceability limit states Type II does not exceed 7 MPa.

It has been assumed that the hardness of the rubber is 50° Shore A (i.e., not less than the 55 ± 5° Shore A given in Table A4 — 1). The compression characteristics have been calculated from data given in Reference 10 and can be conveniently described by the expression:

$$\text{average pressure} = 0.22\,S^2\ \text{MPa}$$

The performance of plain bearings depends on the friction developed between the bearing and the bearing seat or the supported member, since the friction force acts to restrain the bearing and limits the tendency of the bearing to squeeze out. The requirements of Clause 4 — 6.5.3.5 are based on the minimum frictional force, as is the case when the contact surfaces are rubber and steel.

Confined Elastomer Bearings **C4 — 6.6**

The shift in the centre of pressure is limited to 3% in order to prevent excessive deformation of the elastomer. Confined elastomer bearings without lubrication between the elastomer and its contact surfaces should not be used unless approved.

Structure Articulation

C4 — 7

A complete and effective system of structure articulation, based on all identified movements of the structure, is essential. The following methods of providing security and restraint of the superstructure may be used:

a) anchor bolts,
b) prestressing systems,
c) concrete terminal blocks, and/or
d) bearings specifically designed to provide restraint in the required directions.

Methods of computing movements are discussed in Commentary Clause C4 — 4.

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C5 Accessories

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Scope

C5 — 1 The Code does not define criteria for illumination levels, sign message presentation or message legibility. Where criteria for location and clearances are given, these criteria were determined on the basis of preserving the structural integrity of the supports without diminishing the function of the luminaires, signals, or signs. Where applicable, legal requirements or recognized authoritative requirements have been quoted and are identified as such in this Commentary. Differences between current Ministry practices and the requirements of the Code are given in this Commentary.

Although not covered in the Code, small signs designating centre lanes for left turns only or for use at specific periods are sometimes carried on cables spanning between guyed poles. Such supports should include cables attached to the lower portion of the signs so as to reduce sway. This type of support will be covered in a future edition of the Code when Ministry criteria for their use are more firmly established.

The use of barrier walls to attenuate unwanted visual and/or acoustical effects is becoming commonplace. The design requirements for footings for these structures and for sign supports are similar and hence are included in this section.

General Features of Supports

C5 — 4

General

C5 — 4.1

C5 — 4.1.1 This illustrates current practices.

C5 — 4.1.2 Clean simple lines are generally obtained by eliminating struts, braces etc., or by reducing them to a minimum. Safety considerations should not, however, be neglected in improving the aesthetic appearance of supports vulnerable to impact from traffic. The attention given to producing supporting structures of pleasing appearance should be consistent with the effort given to the aesthetic design of other physical features of the highway.

C5 — 4.1.3 Reducing the number of supports helps to minimize cluttering of the roadside environment and at the same time improves safety by reducing the number of potential hazards.

Location of Supports

C5 — 4.2

C5 — 4.2.1 Figure 5 — 4.2.1 provides minimum criteria. Clause 5 — 4.2.1 calls for optimization of safety in addition to meeting minimum requirements.

C5 — 4.2.2 For high-speed roadways, protection or breakaway supports are considered essential within the 10 m limit. Wherever possible, advantage should be taken of guiderail required for other purposes. Since guiderail is generally less safe than an open escape area, its addition to roadside areas should be avoided if possible.

C5 — 4.2.3 The use of guiderail on lower-speed roadways (e.g., city streets) is not considered practicable. Breakaway supports are marginally advantageous at low and intermediate speeds but are not specified since their use near sidewalk areas and signalized intersections must remain a judgemental issue.

Breakaway supports are mandatory for unprotected roadside sign and luminaire supports beside high-speed roadways and not beside lower-speed roadways because at high speeds:

- the potential danger from collision with non-breakaway supports is greater;
- breakaway mechanisms are actuated more effectively; and
- the probability of pedestrian involvement is much lower.

The 10 m dimension from edge of pavement is a somewhat arbitrary figure since it was originally derived from a study of collisions with roadside hazards on level terrain at a posted speed of 96 km/h.

C5 — 4.2.4 Guiderail placed for the sole purpose of protecting supports, other than overhead sign and cantilever sign supports, would itself constitute a greater hazard than breakaway supports.

C5 — 4.2.5 Since a sign support falling across a highway would constitute a hazard greater than that of protective guiderail, this clause forbids breakaway supports for overhead and cantilever signs.

C5 — 4.2.6 On bridges carrying vehicular traffic, advantage should be taken of the available protection. Supports for overhead signs spanning across bridge decks require the protection of a rigid railing because a support falling across the deck would be extremely hazardous.

C5 — 4.2.7 Objects located in the gore area are extremely vulnerable to impact from errant vehicles and should be avoided if at all possible; however, where a major obstruction such as the end of a barrier wall or guiderail is unavoidably located in the gore, supports are permissible. In such a case, an emergency absorbing crash cushion should be provided, particularly on high-speed roadways.

Vertical Clearance

C5 — 4.3

C5 — 4.3.1 **Vertical Clearance for Sign Supports**

C5 — 4.3.1.1 Greater vertical clearances are required for sign bridges than for overpass structures, so that high loads will strike the structures of heavy design first, thereby lessening the chance of major collision damage to the structurally weaker overhead sign support structures. Exceptions may be made for high structures with a large vertical clearance.

C5 — 4.3.1.2 Lane designation arrow signs, used to indicate the lanes to which a sign message applies, are most effective when placed below the sign. In order to prevent inordinately high supports where these lane designation signs are required, Clause 5 — 4.3.1.2 permits infringement on the 5.3 m clearance, provided that no significant force is transferred to the structural support or signboard when the arrow sign is struck. The light construction of these arrow signs makes them susceptible to damage due to impact from flapping tarpaulins on trucks which would otherwise cause no damage. Such damage, however, is usually minimal and the arrow sign may normally be straightened by hand without removal.

C5 — 4.3.2 **Vertical Clearance for Luminaire Supports and Traffic Signal Supports**

C5 — 4.3.2.1 The minimum vertical clearance is taken from Reference 1. Current Ministry policy is to provide a clearance of 9 m.

C5 — 4.3.2.2 The minimum vertical clearance is taken from Reference 2. Current Ministry policy is to provide a clearance of 5 m.

Limit States for Supports

C5 — 5

Ultimate Limit States Design

C5 — 5.1 Due to changing luminaire technology and lighting criteria, some poles are required, in the course of their service life, to support luminaire types which are heavier than those originally intended. For ultimate limit state design, the heaviest and largest luminaires likely to be installed during the life of the support should be considered.

Changes in sign configuration on existing supports are not uncommon, with additional signs or larger signs becoming required due to the development of new policies or traffic conditions.

For ultimate limit state design it is Ministry policy to design all overhead sign supports to accommodate a 2.75 × 15.25 m sign (42 m^2) for spans of 18.3 m and greater. This theoretical sign is placed on the support to provide the maximum response to dead loads and wind loads. For spans of less than 18.3 m, the sign length (L) is reduced to:

$$L = \text{Span Length} - 3.0\ \text{m}$$

and the sign is assumed to be placed symmetrically about the centreline of the span.

Sign supports other than overhead sign supports are normally designed for the actual sign sizes to be used and the support is replaced or modified if additional sign area or weight is to be added. With the advent of large diagrammatic signs and changeable message signs, these Ministry policies will have to be reviewed.

Sections 8 and 9 have clauses making specific reference to pole type supports and give general requirements for the design of reinforced concrete and prestressed concrete members.

Section 10 gives requirements for compact closed sections and for compact and non-compact open sections. Provisions for bolted and welded connections are also given.

Section 13 gives requirements for timber members and connections for these members.

C5 — 5.1.1 The CSA Standard, S157-M, *Strength Design in Aluminum*, has been drafted in limit states format and should be used for the design of aluminum structures; however, the load factors specified in this standard are not compatible with the Code. This clause is written assuming that the standard will be published by the publication date of the Code or shortly thereafter.

C5 — 5.1.2 **Castings**

CSA Standard S157-M, *Strength Design in Aluminum*, does not cover the design of aluminum castings.

Accurate predictions of the strength of castings are often difficult and uneconomical to determine analytically.

Testing is felt to provide an acceptable assessment of the strength of a casting.

To account for the greater variability in the quality of cast material, a reduced performance factor is used to determine the test loading.

C5 — 5.1.3 **Anchor Bolts**

Anchor bolts which are designed to be stronger than the anchored components reduce maintenance costs by increasing the possibility that the anchorage can be reused after impact with a vehicle.

Serviceability Limit States Design

C5 — 5.2

C5 — 5.2.1 **Serviceability Limit State Type I**

Section 2 describes conditions under which vibration may occur in supports, and presents expressions for arriving at corresponding stress ranges.

Section 11 gives permissible fatigue stress ranges for different types of weld details and should be used rather than those specified in CSA S157-M, *Strength Design in Aluminum.*

The difficulties of conducting a valid fatigue test on castings are recognized.

It is felt that a casting which withstands the static loading test called for in Section 5 — 5.1.2 and which is designed with proper consideration of the factors influencing fatigue life, will be acceptable.

Breakaway Supports

C5 — 6

Usage and Function

C5 — 6.1

Actual performance criteria for supports are not given because, although data are available from various tests, the values of the various parameters are not considered sufficiently specific to be included in the Code.

Reference 3 states that:

> "Satisfactory dynamic performance is indicated when the maximum change of momentum for a standard 2250 lb. (1020 kg) vehicle, or its equivalent, striking a breakaway support at speeds from 20 mph to 60 mph (32 km/h to 96 km/h) does not exceed 1100 lb.sec. (4893 N·sec.) but desirably does not exceed 750 lb.sec. (3336 N·sec.)."
>
> (Note: N·sec equals kg·m/s).

This represents a vehicle weight greatly reduced from that used in most vehicle crash testing (1816 kg, 4000 lb) and a lower change in momentum criterion than has been called for in the past. The reasons for these reductions are, respectively, a trend towards smaller cars and improvements in breakaway base technology.

The commentary in Reference 3 explains why test procedures are not defined more precisely and discusses at some length the factors which should be considered in arriving at acceptance criteria and methods of testing.

Vertical Clearance to Breakaway Roadside Sign Supports

C5 — 6.2

C5 — 6.2.1

This clause is based on Ministry practice. It is intended to provide sufficient clearance for the average car to pass below the sign without dictating a sign elevation too high for optimum legibility. Roadside signs normally depend on reflection from headlights and therefore should be kept as close to eye level as possible to ensure night time legibility. It would be unreasonable to reduce the clearance for cars passing under the signs or to cause signs to be higher than would otherwise be necessary by allowing horizontal support members below the lowest edge of the signs.

C5 — 6.2.2

This clause in effect imposes a restriction only for locations where the ground slopes down towards the roadway. In such cases, it is reasonable to assume that any errant vehicle would have its wheels on the ground and a clearance slightly less than that prescribed in Clause 5 — 6.2.1 is permissible.

The 1.35 m clearence is an absolute minimum to permit the support to function as a breakaway member; a desirable clearance to the groundline of 1.50 m should be provided wherever possible. This is particularly important for the support column nearest traffic.

It is Ministry policy to maintain a horizontal clearance of 6.10 m from the edge of pavement to the nearest edge of the sign.

Placement of Footings for Breakaway Roadside Sign Supports

C5 — 6.3

C5 — 6.3.1 The protrusion of concrete footings above groundline is limited to 75 mm in order to prevent the footing concrete and the portions of the support permanently attached to it from snagging on the underside of the vehicle.

C5 — 6.3.2 The purpose of this clause is to meet the requirements of Clause 5 — 6.3.1 while providing for drainage away from the footing.

C5 — 6.3.3 Water ponding on the concrete surface could cause corrosion of metal, and concrete deterioration.

Details for Breakaway Roadside Sign Supports

C5 — 6.4 The provisions of this clause are self-explanatory. Reference 3 implies the use of (b) or (c) as alternatives but Ministry policy has required both.

The Ministry's current detail for developing a fracture plane in the 140 × 140 mm timber posts is two 40 mm holes drilled on the centreline of the post and placed perpendicular to traffic — one, 150 mm above the ground and the other, 300 mm above. In larger members, 50 mm holes are drilled.

References 4 and 5 also provide relevant information and an illustration of the breakaway action.

Spacing of Breakaway Sign Support Posts

C5 — 6.5 The minimum spacing of 2.40 m for two-column supports reduces the probability of a passenger car striking more than one column except in a sideways collision. The commentary on Reference 3 discusses sideways collisions.

The reduced permissible spacing of 1.50 m for multi-column supports is given in order to avoid the necessity for very heavy columns for larger signs. A spacing of 2.40 m should still be used where practicable. When a smaller spacing is used, tests will be carried out assuming that two columns may be struck simultaneously.

Details for Breakaway Luminaire Supports

C5 — 6.6

C5 — 6.6.1 and C5 — 6.6.2 The maximum height restriction for the plane of fracture or slip is included to prevent permanently fixed portions of the support from snagging on the underside of the vehicle.

Placement of Footings for Breakaway Luminaire Supports

C5 — 6.7 See Commentary Clauses C5 — 6.3.1, C5 — 6.3.2 and C5 — 6.3.3.

Foundations for Supports

C5 — 7

Soil Data

C5 — 7.1

C5 — 7.1.1 The installations for which individual soils investigations are specified are larger types of supports which are:

- subject to greater wind loads;
- greater in cost than other supports to install or replace;
- more likely to cause greater damage in case of failure.

The 40 m² area of sign represents the size of sign which the Ministry has been operating successfully for some 15 years without individual soil investigations.

C5 — 7.1.2 Foundations for smaller supports may be designed using reliable information obtained from other sources. These types of supports are often designed with standard foundations for all but the worst possible soils conditions or for wide ranges of soil types.

Foundation Stability and Design

C5 — 7.2

C5 — 7.2.1 Spread footings and piled footings for supports are subject to the same requirements as those of other structures.

C5 — 7.2.2 Widened tops or caps are often used for truss type supports or pole type supports with large bolt circle diameters. They are also used when the upper layer of soil has a poor bearing capacity.

Carrying the bottom of the cap below frost level is usually the best solution, but where this is not practicable (e.g., in areas of extreme frost penetration) other solutions might be found.

In soils susceptible to frost heaving, the cap should always be carried below frost.

C5 — 7.2.3 Reference 6 shows the point of maximum moment in the post to be at one-third the depth of embedment and it is Ministry policy to make this assumption for the serviceability limit state Type II.

C5 — 7.2.4 Earth around unencased portions of embedded posts is usually poorly consolidated and contributes little support to the posts.

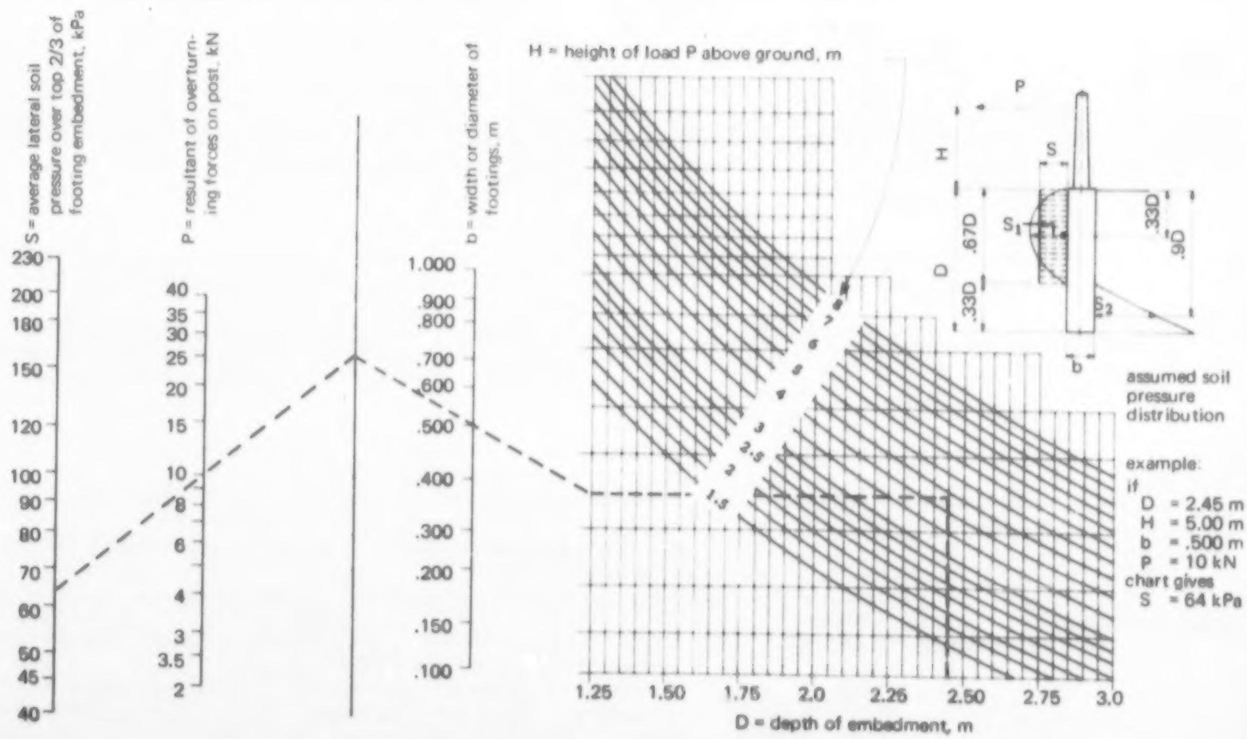
C5 — 7.2.5 The design of "pole type" foundations is governed at the serviceability limit state by restricting movement of the foundation at the groundline within acceptable limits. This is achieved by limiting lateral soil pressures. The nomograph in Figure C5 — 7.2.5, "Chart for Embedment of Posts with Overturning Loads," provides a convenient solution to Equation 5 — 7(3). The chart is very similar to the chart in Reference 3. Equation 5 — 7(3) is taken from Reference 7 and may be readily derived by equating moments about the assumed resultant of the lower stress block as illustrated in Figure C5 — 7.2.5. The equation is based on the assumption that rotation takes place about a point situated at two-thirds of the depth of the foundation.

The expressions for S_1 and S_2 are derived by re-arranging the ratios for S/S_1 and S/S_2 given in Reference 7.

Pole type foundations with parameters outside the limits of the chart may be designed using Equation 5 — 7(3) and the procedure given in Reference 7, checking the stresses in the soil at 0.33 D and at 0.67 D below ground.

The method has been used successfully in Ministry practice for many years.

Figure **Chart for Embedment of Posts with Overturning Loads** **C5 — 7.2.5**



example:
if
D = 2.45 m
H = 5.00 m
b = .500 m
P = 10 kN
chart gives
S = 64 kPa

Detailing of Supports — C5 — 8

Corrosion Prevention — C5 — 8.1

C5 — 8.1.1 The methods of protection listed are acceptable for those steel roadside supports which are readily accessible for maintenance.

It is Ministry policy to hot-dip steel support components wherever possible, because of the extremely aggressive nature of the roadside environment.

The wide selection of steels designated as "stainless steel" offer varying degrees of resistance to corrosion and rusting, and the most suitable should be selected by the designer.

C5 — 8.1.2 Steel sign supports which extend over the travelled portion of the highway, except for stainless steel components, should be hot-dip galvanized in order to reduce the possibility of structural failure due to corrosion, and also to reduce maintenance operations which would interfere with traffic.

Steel components in breakaway slip devices should be galvanized to prevent binding due to rust which would interfere with the operation of the device.

Steel supports mounted on bridges, retaining walls, or other concrete structures should be galvanized to avoid staining of the concrete by rust.

C5 — 8.1.5 Coal tar epoxy paint protects embedded poles from aggressive constituents of the soil or concrete. This requirement is based on Ministry practice.

C5 — 8.1.7 Galvanic action will occur between aluminum and steel unless the steel is stainless or hot-dip galvanized.

C5 — 8.1.8 Experience with Ministry supports has shown that aluminum fasteners and other protective coatings for steel fasteners are unsuitable for connections essential to the structural integrity of the support.

Materials and Mimimum Thicknesses — C5 — 8.2

The minimum thicknesses specified are the minimum thicknesses currently used by the Ministry.

C5 — 8.2.1 **Steel**

C5 — 8.2.1.1 Due to the dynamic nature of the loads imposed on supports, steel used for load-carrying members requires improved resistance to brittle fracture.

C5 — 8.2.2 **Aluminum**

C5 — 8.2.2.1 The properties of aluminum alloys allowed have been shown to be reliable.

Camber — C5 — 8.3

Camber is provided in order to avoid the appearance of sag in the horizontal members.

C5 — 8.3.3 The upward inclination avoids the optical illusion of the luminaire sagging at the end of the bracket and provides an allowance for poor fit.

Connection Details — C5 — 8.4

C5 — 8.4.1 **Structural Steel**

C5 — 8.4.1.2 Circumferential welds in tubular members of supports are subject to large bending moments. Complete penetration butt welds are specified in order that the maximum capacity in bending in the welded joint can be achieved.

C5 — 8.4.1.3 For pole shaft to base plate connections in luminaire supports and/or traffic signal supports, socket type connections with two fillet welds, have been used successfully for many years. Gusset plates are often used in conjunction with this type of connection.

C5 — 8.4.2 **Aluminum**

C5 — 8.4.2.1 CSA Standard S157-M, *Strength Design in Aluminum*, identifies areas in aluminum design that require special consideration.

Anchor Bolts for Supports — C5 — 8.5

This clause ensures that the full strength of the bolt can be utilized. During torquing for pretensioning, bolts are subject to very high pullout forces.

Anchor bolts fabricated from reinforcing steel have been susceptible to failure due to brittle fracture or fatigue.

Bridge Railings — C5 — 9

Railing Material — C5 — 9.2

Concrete, steel, aluminum and wood have been used in the past by the Ministry for both combination and pedestrian railings on bridges.

It is difficult to design connections for wood railing components which are able to withstand traffic loading, and the use of wood for such components is not recommended except where necessary to suit the environment.

The requirement for a minimum tested elongation of 20% is double that of Reference 8. The 20% is met by both the cast steel and cast aluminum posts currently approved by the Ministry and is therefore considered reasonable. Posts with specified elongation of more than 20% have occasionally failed in a non-ductile manner and the increase in required elongation is therefore considered essential. The elongation is specified as being measured in cast coupons since the tested elongation of samples taken from actual castings varies widely and invariably gives results lower than those from cast test coupons. Testing specifications permit a large reduction in elongation requirements for samples from castings.

Railing Design **C5 — 9.3**

C5 — 9.3.1 **Combination Railings**

C5 — 9.3.1.1 This provision is intended to prevent errant vehicles from snagging on posts or other protrusions and being re-directed in an unsatisfactory manner.

C5 — 9.3.1.2 Protrusions and depressions not greater than the limits stated would have insignificant effects on vehicle behaviour and are often unavoidable in practice.

C5 — 9.3.1.3 Locations where rail members are not capable of resisting flexure provide areas where a vehicle could partially or completely penetrate the railing and be subjected to higher decelerations by colliding with rigid components.

C5 — 9.3.2 **Pedestrian Railings**

A smooth continuous upper rail as specified is necessary to prevent injury to the hands of pedestrians using the rail.

Protrusions or depressions within the prescribed limits are often unavoidable but are acceptable provided that their edges are suitably finished. Top rails greater than 90 mm in diameter are inconvenient for a pedestrian to grasp in normal or emergency use.

Application of Loads **C5 — 9.4**

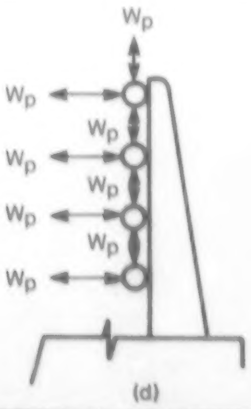
C5 — 9.4.1 **Barrier Walls**

Applying the traffic load P at the top of the barrier produces the maximum response in the wall. Section 7 presents a method for determining distribution of a concentrated load through a slab supported along one edge.

C5 — 9.4.3 to **C5 — 9.4.6** The application of the pedestrian and traffic loads to rails, connections, and posts in various directions and the fractions of the pedestrian load W_p to be applied, parallel the current requirements of References 8 and 9 and current Ministry practice with the following exceptions.

- Barrier wall minimum heights are specified in Section 1 such that the walls will be designed to take all of the traffic load. Therefore, no traffic load is applied to metal railings on top of barrier walls.
- Traffic rails are defined in Section 1 as taking all the traffic loads for railings as shown in Figure 5 — 9.4.1(a). Rails outside the specified zone are considered to be not subject to traffic loads and are therefore excluded from participating in the distribution of these loads. The provisions of Section 1, in effect, require that at least one rail designed as a pedestrian rail be placed above and below the traffic rails.
- For railings of the type shown in Figure 5 — 9.4.1(a), pedestrian loads on the posts are not considered to act simultaneously with the traffic loads and should be neglected.

Figure **Example of Pedestrian Railing** **C5 — 9.4.3**



C5 — 9.4.3 **Pedestrian Rails and Attachments**

Figure C5 — 9.4.3 gives an example of the most commonly used type of pedestrian railing, showing the application of the pedestrian loads to the rails.

Anchor Bolts for Posts **C5 — 9.5** See Commentary Clause C5 — 5.1.3.

Bridge Deck Drains

C5 — 10

Inlet Dimensions

C5 — 10.1

C5 — 10.1.1 Drains with inlets 150 mm in diameter or least dimension have proven to be ineffective. The 200 mm dimension given is a minimum; larger inlets should be used where possible.

C5 — 10.1.2 For the safety of pedestrians, one dimension of every opening should be not more than 75 mm. Cyclists find openings greater than 75 mm measured in their direction of travel uncomfortable and openings substantially greater may act as ruts for tires and cause loss of control.

Deck Drain Placement

C5 — 10.2

C5 — 10.2.1 The inlet should be dished slightly below deck surface but the dishing should not be deep enough nor steep enough to cause a serious bump for traffic on the bridge. Also, dishing in excess of 25 mm may lead to reinforcement cover problems on exposed concrete decks or a very thin wearing surface course.

C5 — 10.2.2 Recessing the drain partially into the curb face or barrier wall face may increase the efficiency at the inlet provided the length of the drain is sufficient to give the water time to flow into the recess.

Design of Drain Inlets

C5 — 10.3

C5 — 10.3.2 For drain inlets smaller than the tire print, the UDL will apply over the full inlet area; for larger inlets the UDL should be placed to produce maximum response.

Corrosion Protection

C5 — 10.4

C5 — 10.4.1 Ministry experience has shown hot-dip galvanizing to be the most cost-effective method of preventing rust in structural steel in the roadside environment.

C5 — 10.4.2 Downpipes should be made from atmospheric corrosion resistant steel to match girders made from such steel. However, the inlet and all non-exposed portions should be hot-dip galvanized to reduce corrosion from concentrated salt attack.

References

[1] American National Standards Institute, *Illumination Engineering Code*, ANSI/IES RP8 1977.
[2] Highway Traffic Act, Province of Ontario.
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[6] Dunham, C.W., *Foundation of Structures*, McGraw-Hill Company.
[7] Ivey, D.L. and Hawkins, L., "Signboard Footings to Resist Wind," *Civil Engineering*, ASCE, 1966.
[8] "Interim Specification, Bridges," AASHTO 1975.
[9] "Design of Highway Bridges," CSA S6-1974.

C6 Substructures and Retaining Walls

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Scope

C6 — 1

The limit states design philosophy has been accepted in structural design for some time and guidance for its usage is available from a number of design specifications. By contrast, the application of limit states design, or even load factor design, to substructures has been quite limited. Codes which specify limit states for structural elements have paid little regard to its usage in substructures and foundations elements. Thus, contradictions have arisen in which two sets of loads have been used — factored loads for superstructure design, and unfactored loads for substructure design. This also means that concrete elements in footings, piles, and retaining walls which are designed to develop ultimate strength against factored loads are not checked for soil failure with these same loads. In most design applications today, the checking of soil failure is still carried out with working stress concepts, a note-worthy exception being the *Danish Code of Practice for Foundations Engineering* [21]. A further difficulty in extending the limit states philosophy to foundation design has been the lack of uniform practices, which are commonly accepted as safety checks. Thus, the required process of calibrating load and resistance factors is rendered difficult by the lack of precise standards governing present-day criteria. As a consequence, Section 6 has set out to establish uniform analysis and design procedures, and to provide the safety factors for the limit states format.

Code requirements must provide against failure under extreme loads and also against excessive deformation under working loads. The latter (denoted here as serviceability checks) will be treated as in the past, with deformations being calculated for the most part using unfactored loads. Safety checks against failure are to be carried out with the load and resistance factors specified in Section 6. Because computations of strength of earthwork, earth-retaining structures, and foundations involve more approximations and uncertainties than for man-made materials, the applied margin of safety must be greater than in superstructure design.

In the calibration process relating to a limit state, there are two paths that are open to the Code writer. The first is to calculate an overall ultimate strength characteristic, such as ultimate bearing capacity, and multiply it by a performance factor. For example, if the current safety margin is established to be 3.0 compared with working loads, this factor would be reduced in the limit state format because the loads are already multiplied by a factor greater than one. This approach to calibration has been used in Section 6 for piles. The second is to apply strength factors directly to soil strength properties. These properties include cohesion (c), and angle of internal friction (ϕ).

The second approach is more sophisticated since it allows the strength factors to be related directly to the uncertainties of the material properties which influence strength. The disadvantage is that it is not consistent with the application to structural elements of performance factors which consider overall performance, such as flexural resistance, rather than material properties, such as f'_c and F_y. Nevertheless, it was found useful to introduce such strength factors as F_c and F_ϕ.

Definitions

C6 — 2

Factored Bearing Capacity at Ultimate Limit States — The definition uses the phrase "breaking into the soil or rock" to describe the mode of failure at the time the bearing capacity of the soil or rock is exceeded. The three principal modes of shear failure under a foundation (as described in Reference 54) are: general shear failure, local shear failure, and punching shear failure. The failure referred to in the Code is closest to punching shear failure.

Bin-Type Retaining Wall — The terms "bin-type retaining walls" or "bin-walls" have been used to cover retaining walls constructed of components fabricated from metal and are usually referred to as bin-walls. Walls fabricated from precast concrete or wood are usually referred to as crib-walls. For purposes of Section 6, crib-walls are considered as bin-type retaining walls or bin-walls.

Deep and Shallow Foundations — Foundations are often classified as shallow or deep depending on the depth at which the load is transferred to the soil. Thus, "deep" foundation, is a relative term. According to Terzaghi [49], a shallow foundation is one in which the depth to the bottom of the footing is less than or equal to the least footing dimension. Others have considered a shallow foundation to be shallow if the depth is $\leq$ 4 or 5 times footing width.

The definition for a "shallow" foundation as given in Reference 18 states that the depth of the bearing area below the adjacent ground is usually about equal to or less than the width of the bearing area, and vertical loads on the sides of the foundation due to adhesion or friction may normally be neglected. Shallow foundations include common footing types such as slabs, rafts, spread footings, strip footings, pads, mats, and sills. The definition for a "deep" foundation as given in Reference 18 states that the loads are transferred by end-bearing to a soil or rock at considerable depth below the structure or by adhesion or friction, or both, in the soil or rock in which it is placed. Piles are the most common type of deep foundation.

In Section 6, the terms "unit", or "foundation unit" are used. For example, "friction unit" means a "friction pile"; "end-bearing unit" means an "end-bearing pile".

Dynamic Impedance — The capability of a deep foundation unit to transmit longitudinal force during driving. This is, in effect, a measure of the ability of the unit to develop the required capacity as a consequence of its being driven into the ground.

Limit States C6—4

Serviceability Limit States C6—4.1

The estimated total and differential movements should be such that:

a) the vertical clearances are not reduced below those specified in Section 1;
b) the ride quality is not reduced below an acceptable level;
c) the structure can accommodate the deformations without structural distress or excessive maintenance;
d) the appearance of the structure is not affected beyond an acceptable level.

Ultimate Limit States C6—4.2

The substructure of a bridge or a retaining wall should be designed to provide safety against failure in the soil as well as in the structure itself. The bearing capacity of foundations at ultimate limit states should be computed using factored soil strength parameters or strength factors in the case where pile load tests have been carried out.

Subsurface Evaluation and Parameters C6—5

Subsurface Evaluation C6—5.1

The subsurface investigation is the first and most important step in any foundation design. Such an investigation should be carried out for any structure before design is undertaken. It is important that subsurface investigation be carried out under the direction of a geotechnical engineer with knowledge and experience in planning and executing such investigations.

Communication between the structural engineer and the geotechnical engineer is essential during each of the planning, design, and construction phases of a project so as to ensure that the requirements of a particular structure at a particular site are met. Only with communication will uneconomical solutions be avoided.

The primary objective of a subsurface investigation is to determine as accurately as may be required:

- the nature and sequence of overburden deposits and rock;
- the groundwater conditions at the site;
- the physical properties of the soils and rock underlying the site;
- the mechanical properties, such as strength and compressibility, of the different soil or rock strata; and
- other information, such as chemical composition of the groundwater and characteristics of foundations of adjacent structures.

Reference 9 will be helpful in planning a subsurface investigation, which should be carried out to such a depth that the entire zone of soil or rock affected by changes caused by the structure or the construction is adequately explored. Reference 32 gives further details on such aspects.

When foundations are extended to the rock surface or into bedrock, it will frequently be found necessary to determine the character and condition of the rock by obtaining core samples. Reference 18 gives further details.

Geotechnical reports based on the data from subsurface investigation are usually referred to for many different purposes during design and construction and, in some cases, after completion of the project.

Soil Strength Parameters C6—5.2

Table 6—5.2 specifies different factors for cohesion and friction, this being consistent with a framework in which parameters which have greater uncertainty have smaller factors. This approach also allows a more precise calibration over a wide range of soil types, which leads to more uniform reliability and a clearer definition of the desired safety checks in each case.

The factors were derived by a calibration procedure such that designs obtained by the new limit state procedures would not differ significantly from designs obtained by working stress methods. The calibration was established using the load factors of Section 2, soil-strength factors presented in the Danish Code [21], and a range of values suggested by Meyerhof, as the starting point. These values were studied with respect to a range of retaining wall dimensions, footing sizes, and soil types as parameters. This calibration study showed that the values contained in Table 6 — 5.2 would produce reasonable and consistent designs. Subsequent studies by Snyder and Moses [48] showed that the values in Table 6 — 5.2 also give consistent values of the safety index, β, when compared to current designs. References 39 and 55 provide additional information.

Bearing Capacity of Shallow Foundations on Rock and Unyielding Soils C6 — 5.3

Due to the fact that computations for ultimate bearing capacity for shallow foundations on rock and unyielding soil yield values which are extremely high and not of practical value for structural design purposes, it was decided to use values selected arbitrarily, but based on values for safe bearing capacity as recommended to the Structural Office by the Foundations Office of the Ministry. These values of safe bearing capacity have been used satisfactorily by the Structural Office in past years.

Loads and Load Effects C6 — 6

Earth Pressure C6 — 6.1

General C6 — 6.1.1

In the computation of earth pressure on a retaining structure, the nature and drainage properties of the backfill material are of primary concern and should always be properly determined. In all cases, free-draining granular material should be used as backfill. In addition, the backfill should be properly compacted. If the backfill were composed of material other than free-draining granular material, the retaining wall would be subjected to forces such as those due to frost heave, swelling, or downward drag caused by subsidence of the soil mass. Such forces cannot be determined reliably by any practical methods. It is not economical to design a retaining wall to withstand these forces unless special studies are carried out in any given case and approved methods of analysis are used.

The earth pressure acting on a retaining wall may range from active earth pressure to passive earth pressure, depending on the displacement characteristics of the wall and the methods of compaction of the backfill. The active earth pressure is the minimum value of lateral earth pressure which a soil mass can exert against a yielding retaining structure. The passive earth pressure is the maximum value of lateral earth pressure which can be mobilized by a structure moving against a soil mass. Earth pressure at-rest is the horizontal effective stress which exists in a soil mass when there is no lateral deformation. Table C6 — 6.1.1 indicates the order of magnitude of wall rotation Y/H, where Y is the displacement of the crest of the wall and H is the effective height of the wall, required to achieve the active and passive earth pressure in dense and loose cohesionless soils.

Table **Wall Rotation Y/H for Active and Passive Earth Pressure** **C6 — 6.1.1**

Type of Backfill Material	Rotation Y/H	
	Active	Passive
Dense Cohesionless	0.5×10^{-3}	2×10^{-3}
Loose Cohesionless	2×10^{-3}	6×10^{-3}

where: Y = the displacement of the crest of the wall, and
H = the effective height of the wall.

In general, a retaining wall can be assumed to be acted on by active earth pressure because of its flexibility. The abutment of a rigid frame structure should be designed to withstand the at-rest earth pressure because the wall is either relatively rigid, or constrained at its crest. If the backfill is subjected to heavy vibratory compaction, the earth pressure may exceed the at-rest value due to the high residual stress locked in the soil mass.

Computation of Earth Pressure C6 — 6.1.2

C6 — 6.1.2.1

The at-rest earth pressure may be evaluated by the theory of elasticity. However, as an approximation, the coefficient of at-rest earth pressure, $(K_o)_f$, for a loose cohesionless soil at the ultimate limit state, may be estimated from the following empirical formula:

$$(K_o)_f = 1 - \sin \phi_f$$

Similarly, for the serviceability limit states the coefficient of earth pressure, K_o, may be estimated from:

$$K_o = 1 - \sin \phi$$

For heavily-compacted soil, much higher earth pressures can occur [15].

The active and the passive earth pressure may be evaluated by the theory of plasticity or by considering the limiting equilibrium of the retained soil mass. One of the methods that is commonly used is Coulomb's Theory. Although Coulomb's Theory is approximate in nature, in the sense that it assumes a planar sliding surface and may not satisfy all the equilibrium conditions, it nevertheless allows the designer to easily take into account the effects of surcharge, sloping ground surface, wall friction, and non-homogeneity of the soil. Moreover, the value of active earth pressure obtained by Coulomb's Theory for cohesionless soil does not differ appreciably from those obtained by more refined and rigorous methods. For passive earth pressure, Coulomb's Theory tends to over-estimate somewhat the magnitude of the pressure.

For ultimate limit states, the active and the passive earth pressure coefficients $(K_a)_f$ and $(K_p)_f$ can be obtained using factored soil strength parameters. For serviceability limit states, the active and the passive earth pressure coefficients can be obtained by using the unfactored soil strength parameter tan ϕ.

C6 — 6.1.2.2

For a simple case where the backfill is approximately level and specified as free-draining granular material and the retaining wall does not exceed 10 m in effective height, the angle of internal friction of the backfill may be assumed to be equal to $\phi = 30°$. The coefficients of active and passive earth pressure for the ultimate limit state and the serviceability limit state may be computed as shown below.

From tan ϕ_f = 0.8 tan ϕ, the modified angle of internal friction ϕ_f, is found to be 24.8°. Hence, the coefficients of earth pressure, in terms of ϕ and ϕ_f, are computed as follows:

	Ultimate Limit States	Serviceability Limit States
Active State	$(K_a)_f = \frac{1 - \sin\phi_f}{1 + \sin\phi_f} = 0.41$	$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.33$
At-Rest State	$(K_o)_f = 1 - \sin\phi_f = 0.58$	$K_o = 1 - \sin\phi = 0.50$
Passive State	$(K_p)_f = \frac{1 + \sin\phi_f}{1 - \sin\phi_f} = 2.44$	$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3.00$

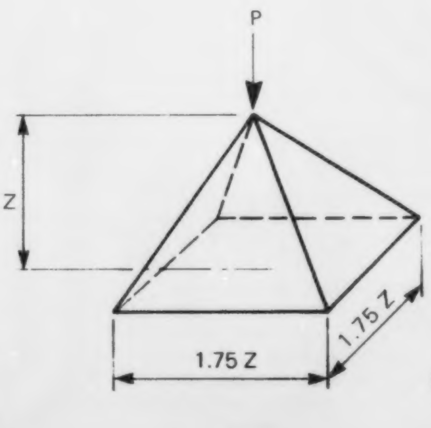
For example, in the case of a cantilever retaining wall, with a height of 6 m, retaining a granular backfill with a unit weight of 20 kN/m³, the total factored active force at the ultimate limit state acting on the wall, per metre length of wall, is equal to:

$$P_a = 1/2(K_a)_f \gamma H^2 \times \text{(Load Factor)}$$
$$= 1/2 \times 0.41 \times 20 \times 6^2 \times 1.25 = 184 \text{ kN/m}$$

Similarly, the total factored passive resistance at the ultimate limit state is equal to:

$$P_p = 1/2 (K_p)_f \gamma H^2 \times \text{(Performance Factor)}$$
$$= 1/2 \times 2.44 \times 20 \times 6^2 \times 0.8 = 703 \text{ kN/m}$$

Figure **Distribution of Wheel Loads Through Fills** **C6 — 6.2.2**



$$P = \frac{P}{(1.75 Z)^2}$$

C6 — 6.1.2.3 Where a structure is retaining a soil mass which has a sloping ground surface, or is subjected to an unusual surcharge, or where heavy vibratory compaction is involved, the pertinent contributing factors should be duly taken into consideration in the computation of earth pressure. In such a situation the earth pressure should be computed by a geotechnical engineer.

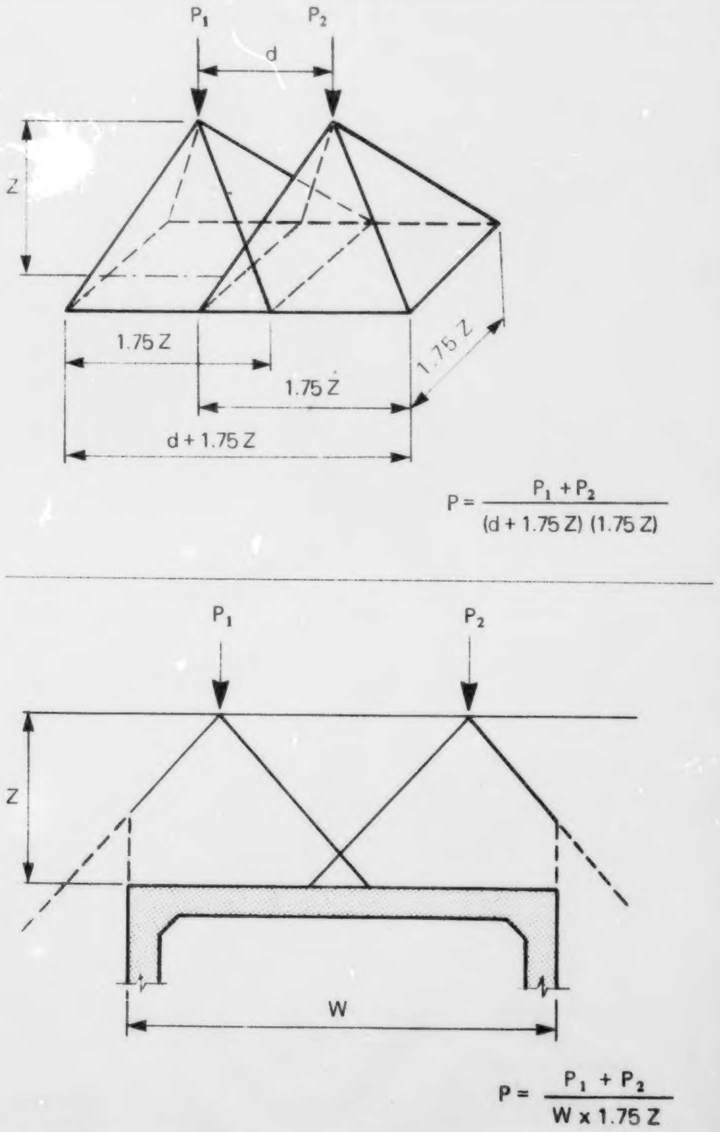
C6 — 6.1.2.4 The surcharge loading specified in this clause provides for the load effect from live load and should be used in the design of all abutment walls, and wingwalls located within the specified distance from the traffic. References 18, 33, 50, 52 and 57 provide additional information.

Distribution of Wheel Loads **C6 — 6.2**

C6 — 6.2.2 Figure C6 — 6.2.2 illustrates the distribution of live loads.

C6 — 6.2.3 Figure C6 — 6.2.3 illustrates the distribution of live loads as specified in this clause, and covers overlap conditions and limits imposed by the boundaries of a structure.

Figure **Distribution of Live Load — Effects of Overlap of Loads and Boundaries of Structure** **C6 — 6.2.3**



$$P = \frac{P_1 + P_2}{(d + 1.75\,Z)\,(1.75\,Z)}$$

$$P = \frac{P_1 + P_2}{W \times 1.75\,Z}$$

C6 — 6.2.4 When the depth of the fill exceeds 2.5 m, the live load acting upon the surface of the structure is very small compared to the total dead load and may be ignored.

C6 — 6.2.5 For small depths of fills (i.e., fills not exceeding 600 mm in depth), wheel loads are normally considered to act as concentrated loads at the surface of the structure for convenience of computations. The depth of fill will be considered to include the pavement and should be measured to the finished riding surface.

Backfill and Drainage **C6 — 6.3**

The backfill behind any structure which is not designed to withstand the full hydrostatic pressure of submerged soil should consist of free-draining granular material, and adequate drainage should be provided to ensure that the fill material will not become saturated and submerged. Drainage may be provided by weepholes and/or perforated pipes.

Water Pressure **C6 — 6.4** As a rule, if the requisite observations of water levels are available, the highest water level should be defined as the level above which the water table has at most a probability of 5% of rising during the expected useful life of the structure. The lowest water level is defined in the corresponding way.

When the water level has been established, hydrostatic water pressures are known with the same accuracy as dead loads and, for this reason, the actual value of the hydrostatic water pressure should be used. Moreover, any other value would lead to similar complications as for dead loads, because the uplift force on a soil or structural element is a part of its effective weight.

When water levels cannot be accurately predicted, or if hydro-dynamic water pressures may occur, the water pressure should be increased by 20% for maximum load effects. This will also be necessary in cases where the stability of a structure depends almost exclusively on dead loads and water pressures.

Figure **Influence of a Structure Footing on an Adjacent Excavation for a New Footing** **C6 — 7.2(c)(i)**

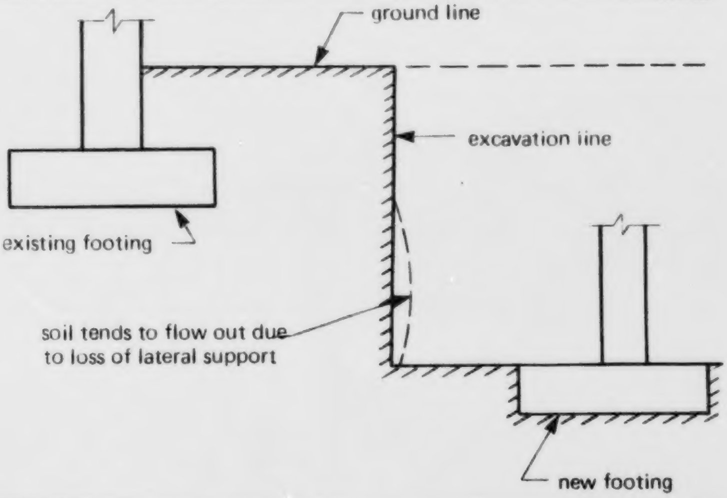
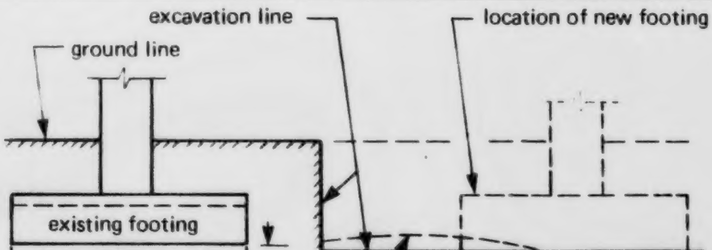

ground line
excavation line
existing footing
soil tends to flow out due to loss of lateral support
new footing

Figure **Settlement Caused by an Adjacent Excavation** **C6 — 7.2(c)(ii)**


excavation line
location of new footing
ground line
existing footing
settlement caused by adjacent soil movement
potential soil "bulge"

Shallow Foundations C6—7

General C6—7.1

Although the requirements of Clause 6—7.1 apply to single footings which support a single column or wall, most of the requirements are also applicable to combined footings and mats which support two or more columns or walls. Additional information concerning the design of combined footings and mats is given in Clause C6—7.3.5.

Depth Criteria C6—7.2

a) Clauses C6—7.3.3.1 and C6—7.3.4 provide information on bearing capacity and compressibility characteristics, respectively.

b) The requirements for scour provision are covered in Section 1.

c) When placing a new shallow foundation unit lower than an adjacent existing unit, there is a possibility that the soil may tend to flow out from beneath the existing foundation, causing movements of the existing structure. This type of problem is difficult to analyse and if such a construction is anticipated, it is recommended that an experienced geotechnical engineer be consulted. In some cases, the problem may be avoided by constructing a sheet pile wall to retain the soil without movement, as illustrated in Figure C6—7.2(c)(i). Removal of overburden adjacent to an existing shallow foundation unit can sometimes result in a ground movement causing settlement of the existing unit and an upward bulging of the soil in the vicinity of the new foundation unit. It is difficult to predict how close an excavation can be carried out without causing distress; this depends on the subsoil conditions and the type of construction. This type of problem is illustrated in Figure C6—7.2(c)(ii).

d) Future dredging or excavation carried out adjacent to a shallow foundation can lead to undermining, resulting in failure or settlement of a structure in the manner described in Clause C6—7.2(c). Accordingly, the possibility of future ground lowering adjacent to a shallow foundation should be considered when determining the depth of the foundation.

e) Reference 18 is helpful in this regard.

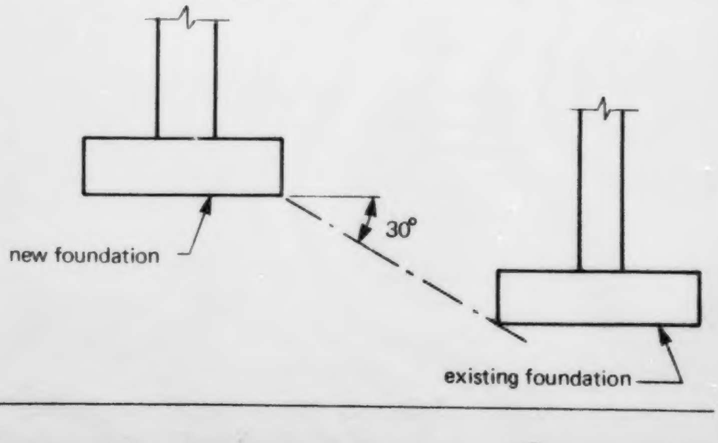
f) For frost action to occur, it is generally accepted that three basic conditions must exist:

- the presence of frost-susceptible soil;
- the availability of water;
- and cooling conditions that cause the soil and water to freeze.

Considerable differences of frost penetration can exist throughout the province and, in some instances, even locally. The bottom of a footing should be located below the lowest frostline that is likely to occur during the life of a structure. The depth of soil cover over the top of a concrete footing should also be sufficient to provide for the difference between the thermal conductivities of the soil and concrete. Where frost protection is marginal or deficient (for example, in the case where a roadway grade adjacent to an existing structure is lowered), consideration should be given to the use of insulation to improve frost protection. References 18 and 19 provide additional information.

g) The extent of volume changes of cohesive soils depends on the moisture content of the soil, which can vary seasonally, and could result in settlement or deformation of the structure. Similarly, soil can dry out from excavation and the resulting change in volume can cause differential settlement of a shallow foundation.

Figure **Recommended Location for New Foundation** **C6—7.2(h)(i)**



Structures founded on such cohesive soils require special construction techniques. Reference 18 provides additional information.

h) i) When a shallow foundation is placed adjacent to an existing foundation which is at a lower elevation, it should be located so that the 30° line from the bottom edge of the new foundation does not intersect the underside of the existing foundation, as shown in Figure C6 — 7.2(h)(i). This is to avoid the possibility of imposing additional loads on the existing foundation.
When a shallow foundation is placed adjacent to an existing foundation which is at a higher elevation, a geotechnical engineer should be consulted for recommendations to avoid the possibility of undermining the existing foundation. See also Commentary Clause C6 — 7.2(c).
   ii) A new foundation can impose additional loads on an underground conduit, depending on the nature of the founding soil and the location of the buried conduit relative to the foundation. As a precaution, the new foundation should be located as shown in Figure C6 — 7.2(h)(ii). Reference 18 provides additional information.

Design C6 — 7.3

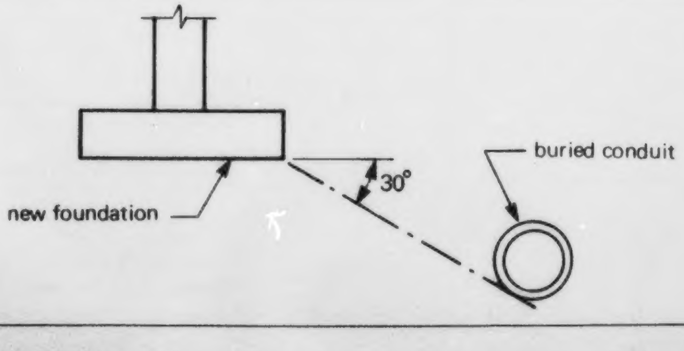
C6 — 7.3.1 General

a) In determining the proportions of a shallow foundation for geotechnical considerations, the designer should consider the following.
   i) At the ultimate limit states, the factored bearing capacity of the founding soil or rock should not be exceeded by the imposed factored loads. The factored bearing capacity at the ultimate limit states should be determined using the factored soil parameters specified in Clause 6 — 5.2. In the case of inclined resultant loads, the factored bearing capacity is reduced as specified in Clause 6 — 7.3.3.5.
   ii) At serviceability limit states Type II, the bearing capacity of the founding soil or rock should not be exceeded by the bearing pressures due to the imposed unfactored loads. The bearing capacity at serviceability limit states Type II should be determined using unfactored soil parameters as specified in Clause 6 — 7.3.4.3.

b) The amount of total, differential, and uneven settlement that can be accommodated by a structure depends on its type of construction and configuration. Structures which are simply supported or have relatively long, slender superstructures can generally tolerate larger differential or uneven settlements than those which are continuous or have short, stiff superstructures. Large settlements of a structure unaccompanied by differential or uneven settlements may not affect the strength and performance of the structure but could result in an unacceptable riding quality or an unacceptable decrease in vertical clearance. The designer should, therefore, consider the capability of a structure to accommodate settlements, and establish the settlements that can be tolerated.

c) Sustained and/or transient loads may have significant effects on the magnitude of settlements of the supporting soil. The following factors should be considered in the computation of the total and differential settlements:
   i) the ratio of sustained load to total load;
   ii) the duration of sustained loads;
   iii) the time interval over which settlement occurs.
The consolidation settlements in cohesive soils are time-dependent and, consequently, transient loads have negligible effect. However, in granular soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place.

Figure **Recommended Location for New Foundation** **C6 — 7.2(h)(ii)**

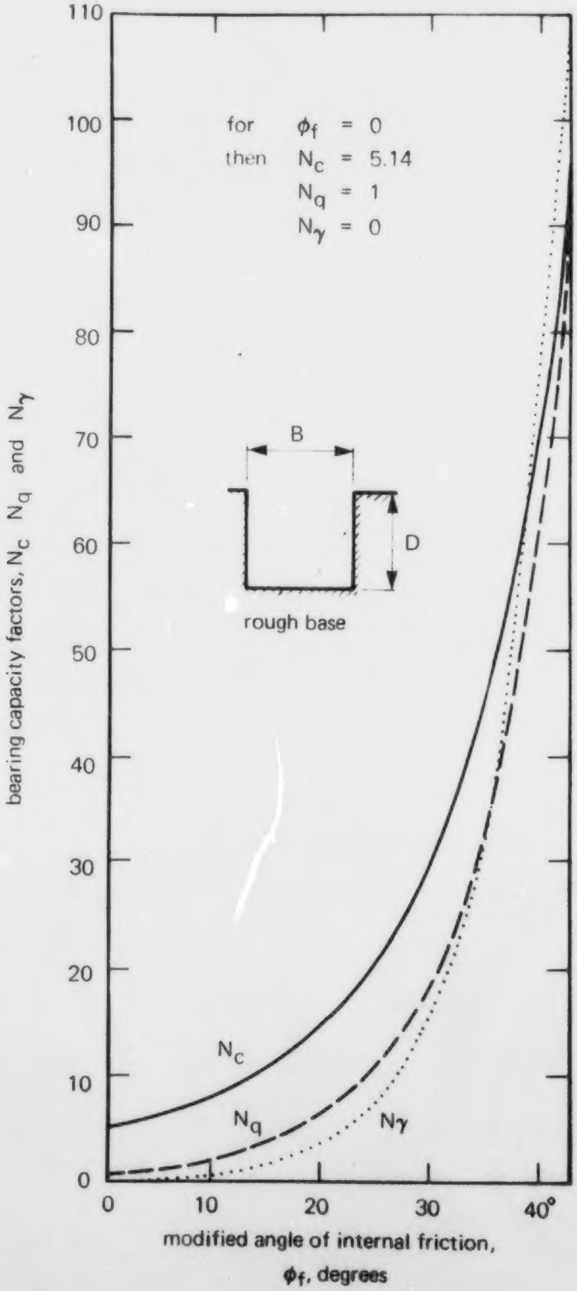


In granular soils, the deformation takes place during construction as the loads are applied. Such deformation can be accommodated by the structure to an extent depending on the type and construction methods.

d) The depth at which a shallow foundation unit is to be founded should be based upon the depth criteria given in Clause 6 — 7.2.

When a shallow footing is located on, or adjacent to a slope, the lack of soil on the down-slope side of the footing tends to reduce its stability. Reference 37 provides useful information in such cases.

Figure **Bearing Capacity Factors, Ultimate Limit States (Ref. 35)** **C6 — 7.3.3.1**



It is often advantageous, and sometimes necessary, to place shallow foundation units on well-compacted earth fills, using controlled methods of construction and materials. Such fills often provide a better foundation base than the original subsoil underlying the fill. The bearing capacity and settlement of the foundation units placed on compacted fills depend on the type of fill material, construction control, and the proximity of the footing to the sloping surface of the fill. These aspects should be investigated by a geotechnical engineer.

C6 — 7.3.2 See Commentary Clauses C6 — 7.3.1(a) and (b).

C6 — 7.3.3

C6 — 7.3.3.1 The factored bearing capacity, q_f, of shallow foundation units at ultimate limit states may be computed from the following equations.

a) For rectangular units with $D/B > 2.5$ placed on cohesive soils, and for all rectangular units placed on granular soils:

$$q_f = c_f N_c + \gamma D N_q + 0.5\gamma B N_\gamma$$

b) For rectangular units with $D/B \leq 2.5$ placed on cohesive soils:

$$q_f = \gamma D + 5c_f\{1 + 0.2(D/B)\}\{1 + 0.2(B/L)\}$$

c) For square units placed on cohesive or granular soils:

$$q_f = 1.2\,c_f N_c + \gamma D N_q + 0.4\gamma B N_\gamma$$

d) For circular units placed on cohesive or granular soils;

$$q_f = 1.2 c_f N_c + \gamma D N_q + 0.6\gamma r N_\gamma$$

The bearing capacity factors N_c, N_q, N_γ should be determined from Figure C6 — 7.3.3.1 using values of ϕ_f obtained by using the soil strength factors as specified in Clause 6 — 5.2 [35].

Figure **Base Under Eccentric Inclined Load at Failure** **C6 — 7.3.3.4(i)**

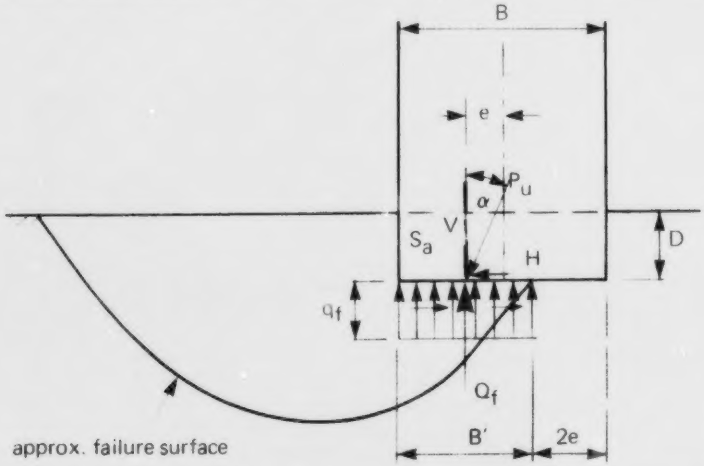
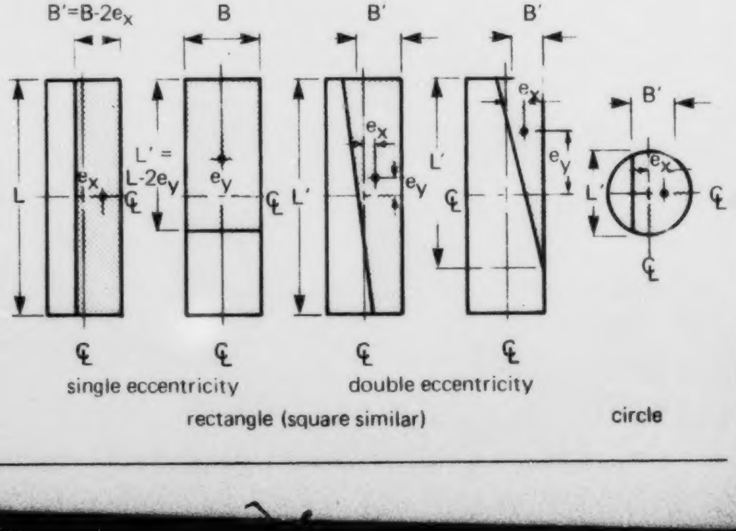


Figure **Effective Contact Area of Foundations with Eccentric Load** **C6 — 7.3.3.4(ii)**



The equations given above apply only to homogeneous soils. If either a layered soil condition or the water table intersect the probable failure surface, the equations above seldom apply. For silts, both drained and undrained conditions should be considered.

C6 — 7.3.3.2

Adequate adhesion at the soil-footing interface (or between the foundation and soil) is assumed as part of the reduction procedure for bearing capacity, Clause 6 — 7.3.3.5. This is not always the case. Designers may use the following to check sliding for cast-in-place footings, where sliding failures occur in the soil.

$$Ac'_f + V\tan\phi_f > H$$

where:
V = the sum of the factored vertical loads acting, kN
H = the sum of tha factored horizontal loads acting, kN

For the initial conditions in clay ($\phi = 0$), where drainage has not occurred, the following applies:

$$Ac'_f > H, \text{ noting that } H/V < 0.4$$

C6 — 7.3.3.4

For spread footings subjected to eccentric loading, the uniform distribution of the soil reaction over the effective area of the footing should be used for the geotechnical analysis of the footing. Here, reference is made to the size of the footing in plan so as to avoid the footing breaking into the soil.

For an eccentricity e of the resultant load P_u on the base of a footing of width B, the effective B' is the actual width less twice the eccentricity, as shown in Figure C6 — 7.3.3.4(i), that is:

$$B' = B - 2e$$

For eccentricity in two directions, the effective area can be more accurately determined in such a way that the resultant of the load passes through its centroid. In this case a straight line of rotation is required as the boundary of the effective area as shown in Figure C6 — 7.3.3.4(ii).

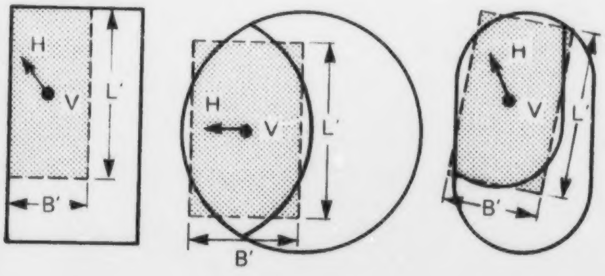
However, in practice it is sufficiently accurate to use an approximate rectangular area as follows:

$$B' = B - 2e_x$$
$$L' = L - 2e_y \text{ and}$$
$$A' = B' \times L'$$

Examples of approximate equivalent effective areas for different footing shapes are indicated in Figures C6 — 7.3.3.4(ii) and C6 — 7.3.3.4(iii) Reference 36 is recommended for consultation.

The square or rectangular contact distribution is a convenience for geotechnical purposes only. Other distributions satisfying equilibrium and the rotation of the rigid footing on a yielding soil are preferred. For other than rectangular or square distributions, the B' value appropriate to the rectangular or square distribution can be used in the bearing capacity equation.

Figure **Equivalent and Effective Foundation Areas** **C6 — 7.3.3.4(iii)**



C6 — 7.3.3.5

Figure C6 — 7.3.3.5 indicates the contact pressure distribution which can be assumed for bearing capacity check at the ultimate limit state under an eccentric inclined load. The bearing capacity check at the ultimate limit states ensures safety against rupture of the underlying soil, and hence overturning [36].

In conjunction with the bearing capacity check at serviceability limit states Type II (which is described in Commentary Clause 6 — 7.3.4), the above check enables footing dimensions to be established.

C6 — 7.3.3.6

In the structural design of a footing for ultimate limit states, the designer must consider two cases of contact pressure distribution. The first case corresponds to the contact pressure distribution due to a yielding soil as prescribed in Clause 6 — 7.3.3.4 and approximates to a uniform pressure distribution over the effective area, A'. The second case corresponds to the contact pressure distribution due to a non-yielding soil which may be considered as a linear contact pressure distribution, assuming a linear elastic response of the soil and a nearly rigid footing. Figure C6 — 7.3.3.6 illustrates the two contact pressure configurations to be considered in the structural design of a footing at ultimate limit states.

The computed capacity, q_f, represents a minimum expected from the soil based on the appropriate factored soil strength parameters including adjustment for load inclination (R_i). The uniform pressure distribution is appropriate for geotechnical analysis (that is, the determination of A') and is also applicable for structural analysis at the ultimate limit states, see Figure C6 — 7.3.3.6(a).

However, structural analysis should also include the case of soil with capacity greater than the calculated minimum, q_f. For these soils, elastic response to factored load is probable (Figure C6 — 7.3.3.6(b)); hence, a linearly varying pressure results from the soil to the footing. In addition, for footings on rock or dense glacial till, the contact pressure at ultimate limit states is more closely represented by the linear configuration rather than the uniform configuration. For the linear varying distribution at the ultimate limit states, the maximum pressure for structural design is not limited to the minimum capacity, q_f, or $R_i \times q_f$.

The requirement to consider two pressure distributions for structural design is new in Edition 2 of this Code.

C6 — 7.3.4

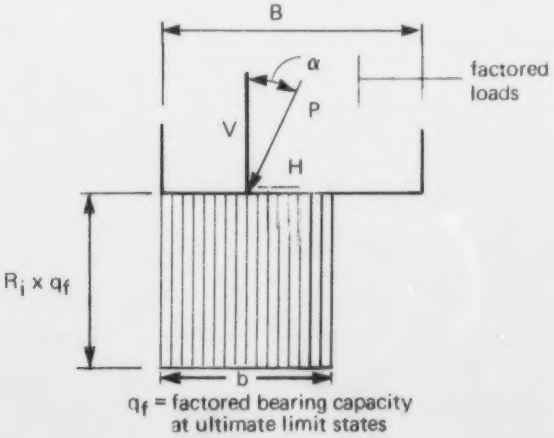
The settlement of a structure is the result of deformation of the underlying subsoil. The deformation arises from elastic distortion, consolidation, general shear deformation, and subsoil collapse due to such factors as mining subsidence.

Elastic deformation is generally small and can usually be neglected. Reference 57 gives further details.

The consolidation settlement of an underlying subsoil is a result of a reduction in pore water pressure and hence is dependent on the permeability of the subsoil.

General shear deformation occurs in underlying subsoil where the shearing stress is excessive. In rock, the shear deformation occurs along planes of discontinuities.

Figure **Bearing Capacity, Ultimate Limit States with Eccentric Inclined Load** **C6 — 7.3.3.5**



q_f = factored bearing capacity at ultimate limit states

Figure **Uniform Pressure Distribution for the Geotechnical Analysis for a Footing, Ultimate Limit States, Eccentric Inclined Load** **C6 – 7.3.3.6(a)**

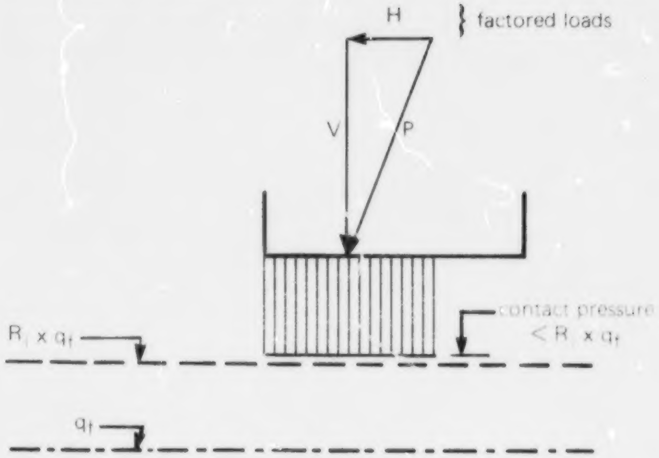

H
factored loads
V
P
contact pressure
$< R_i \times q_f$
$R_i \times q_f$
q_f

Figure **Uniform and Linear Pressure Distributions for the Structural Design of a Footing, Ultimate Limit States, Eccentric Inclined Loading** **C6 – 7.3.3.6(b)**

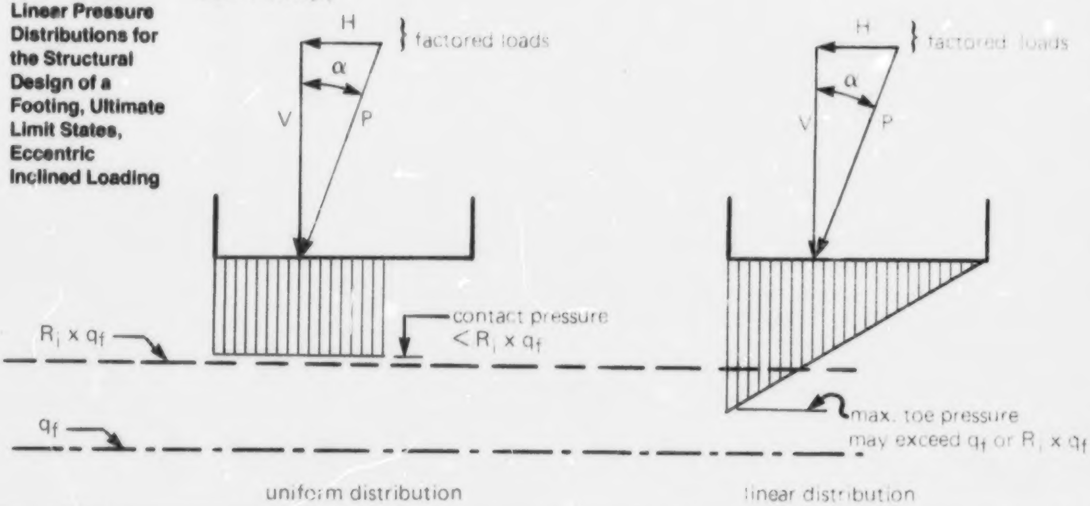
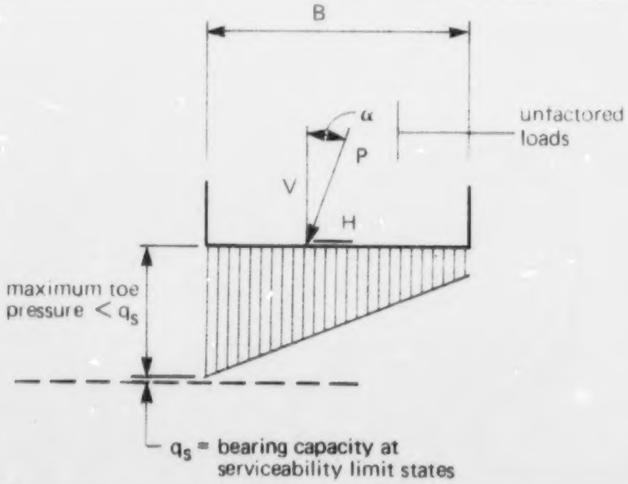

H
factored loads
α
V
P
contact pressure
$< R_i \times q_f$
$R_i \times q_f$
q_f
H
factored loads
α
V
P
max. toe pressure
may exceed q_f or $R_i \times q_f$
uniform distribution
linear distribution

Figure **Bearing Capacity, Serviceability Limit States** **C6 – 7.3.4**


B
α
unfactored
loads
P
V
H
maximum toe
pressure $< q_s$
q_s = bearing capacity at
serviceability limit states

Guidance should be sought from a geotechnical engineer.

Subsoil collapse is a local occurrence and its evaluation is based on regional experience. Guidance should be sought from a geotechnical engineer where appropriate.

In sands and gravels, the permeability is such that consolidation normally takes place during construction. Settlement of sands and gravels is largely due to the rearrangement of particles and hence no rational method is available for the prediction of these settlements. However, a number of empirical and semi-empirical methods have been developed and are outlined in Reference 18. Since correlation between predicted and observed results is not generally good, extreme care should be exercised in applying these methods.

The permeability of fine-grained soils is such that the dissipation of pore pressures is very slow and hence the consolidation is time dependent. Total settlements can be estimated based on laboratory tests and consolidation theories as follows:

$$S = \frac{e_0 - e_1}{1 + e_0} \cdot H$$

where:
S = settlement or change in stratum thickness
H = total thickness of compressible soil stratum
e_0 = initial void ratio
e_1 = void ratio after loading

Time-dependent settlements can be estimated based on laboratory measured permeability coefficients. The related time-dependent consolidation theories and formulae are described fully in Reference 57.

Figure C6—7.3.4 indicates the contact pressure distribution which can be assumed for bearing capacity check at serviceability limit state Type II. This check ensures that tilting and horizontal movements will be within acceptable limits.

C6—7.3.5 **Combined Footings and Mats**

For bridge foundations, combined footings usually consist of footings under two columns or strip footings supporting more than two columns. Mat foundations supporting an array of columns or walls are usually encountered in piers for large or unusual structures on poor soils.

For rigid combined footings and mats, the distribution of soil reactions or pile reactions may be determined in the same manner as for centrally-loaded or eccentrically-loaded single footings. Flexible combined footings and mats may be designed using simplified elastic methods based on the modulus of subgrade reaction. Reference 2 provides criteria which may be used to establish if a footing should be considered rigid or flexible.

Deep Foundations **C6—8**

General **C6—8.1**

Sharing of the load between the deep foundation units and the footing is not permitted. Although some sharing of the load may take place in some cases, there is no simple and practical method of analysis with which to predict the portion of load that can be taken by the footing. Also, the soil under the footing can settle, which transfers the load to the deep foundation units.

Depth Criteria **C6—8.2**

b) The type of unit selected depends on the anticipated settlement of the unit which should be compatible with the amount of settlement that can be tolerated by the structure. An end-bearing foundation unit generally provides the best resistance to load with a minimum of settlement. Friction units are usually subject to settlement and should be used only where the predicted settlements can be tolerated by the structure.

d) A change in ground loading through the removal or addition of soil in the vicinity of a structure can induce additional lateral load onto a deep foundation unit.

e) Where future dredging or excavation is anticipated, the deep foundation units should be carried to such a depth that they can continue to support the imposed loads after removal of material surrounding the units.

f) When deep foundation units are driven adjacent to existing foundations or utilities, precautions should be taken to avoid damage to the existing installation from vibrations during pile driving and ground displacements.

g) The potential for construction difficulties as a result of the water table should be considered, particularly in the case of cast-in-place concrete caissons and pile cap footings.

References 16 and 17 provide additional information.

Design **C6 — 8.3**

C6 — 8.3.3 Factored Axial Capacity at Ultimate Limit State

The capacity of a deep foundation unit may be determined by means of load testing, dynamic methods of analysis, or geotechnical formula analysis. The factored axial capacity of a unit is determined by multiplying the capacity obtained by load testing, or obtained by dynamic methods of analysis, with the appropriate strength factor, or by applying factored soil parameters to the capacity obtained from geotechnical formula analysis.

The capacity determined by load testing is more reliable than the capacity obtained from dynamic methods of analysis. This is reflected by the higher strength factors specified in the case of the load test. The confidence in the reliability of the obtained load test data and the representativeness of the data may vary between routine tests and high-level tests. This is reflected in the separation of the strength factors between routine and high-level tests.

The dynamic methods of analysis normally used are wave equation analysis, dynamic pile driving formulae, and dynamic field measurements. Again, the higher confidence in calculations using field measurements is reflected in a higher strength factor, when the analysis is based on actual field measurements, as opposed to analysis without such measurements.

Where the axial capacity of a deep foundation is not determined in advance of design by load testing or a dynamic method of analysis, Clause 6 — 8.3.3.2 specifies that the factored axial capacity of a unit be computed using a geotechnical formula and applying factored soil parameters as specified in Clause 6 — 5.2.

When a unit is test loaded in the field, the test serves to verify the capacity of the unit as a structural element, as well as its capacity based on geotechnical limitations. In the absence of structural and geotechnical verification by field testing, the factored capacity may be computed using a geotechnical formula such as that given in Clause C6 — 8.3.3.2, provided that the capacity is reduced by the use of the capacity modification factors specified in Table 6 — 8.3.3.2. The strength factors specified in Clause 6 — 8.3.3.1 and the capacity modification factors specified in Table 6 — 8.3.3.2 have been derived by a calibration procedure so that designs conforming with the new limit states design procedures do not differ significantly from designs meeting the current working stress methods.

C6 — 8.3.3.1 a) Load Test

Load testing can be performed as either a routine load test or a high-level load test. A routine test is usually carried out for the purpose of proof testing and involves limited quality control of the test data, a limited number of test piles, and only limited analysis of the test results. A high-level test is usually carried out before finalizing the design and involves highly reliable test data, a testing program and method that are adapted to the particular site conditions and problems, and a detailed analysis of the test data.

- *Test Arrangement and Load Test Methods*

Every load test must be arranged in conformity with the ASTM D-1143 [10]. If the minimum distance values recommended in the ASTM standard are reduced, the reliability and usefulness of the test results could be impaired. The test load is usually applied by means of an hydraulic jack (which is also acting as a load cell measuring the applied load). However, where a higher accuracy and confidence in the test results is needed, such as in high-level testing, a separate load cell has to be used to measure the applied loads.

The measuring of the pile compression by means of a telltale to the pile end, and consequently therefore also the movement of the pile end, will greatly enhance the value of a load test. It should be considered for high-level testing and, wherever possible, also for routine tests on long piles.

Where the objective is to determine the factored bearing capacity of the pile for a limit states design, the quick tests methods are technically preferable over the slow methods.

For methods of testing piles for uplift, see Reference 11.

- *Interpretation of Load Test Results*

A test loading shall preferably be brought to ultimate failure, the exception being a test for proof testing purposes. When failure has not been reached in the test, Q_b shall be taken as the maximum test load applied to the pile.

When evaluating test loading results for ultimate limit states design, the ultimate failure load (i.e., plunging failure load) shall be used rather than a load value determined from a settlement criterion. The failure criterion used should be based on the shape of the load-movement diagram. Such methods are recommended because they provide an understanding of the pile capacity and are in conformity with interpretation criteria used in other civil engineering applications. References 18 and 28 provide information in this regard.

b) **Dynamic Methods of Analysis**

The following dynamic methods of analysis are normally used to determine pile capacity.

- *Wave Equation Analysis*

The one-dimensional wave equation is the application of longitudinal wave transmission to the pile driving process. The method requires the use of a computer program [29, 31].

The wave equation analysis is recognized as one of the major advances of the current state-of-the-art. Although it is highly recommended, it should be considered as a tool among many, and should only be used by a person proficient in the art of pile installation and pile-soil analysis.

- *Dynamic Pile Driving Formulae*

The so-called dynamic pile driving formulae are derived by equating the nominally available or rated energy (not the energy actually delivered) with work performed by the pile, calculated as the static capacity of the pile times the penetration for the blow. Caution must be exercised in the use of dynamic formulae as they are fundamentally incorrect.

- *Dynamic Field Measurements*

The bearing capacity of a single pile can be determined by means of dynamic monitoring using special field instrumentation and methods of analysis. The measurements and the evaluation of the data must be carried out by a person competent in this field.

The advantages of the dynamic monitoring method are that several piles can be tested for the cost of one load test to account for the natural variations of capacity between the piles and that, apart from the capacity estimation, the method also provides a control of hammer efficiency, determines the energy and driving stresses developed in the pile, and, thus, provides data to use for calibration of a wave equation analysis, and to use in the inspection of the installation work.

C6 – 6.3.3.2 Piles derive their load-carrying capacity from point resistance and/or shaft resistance. The relative distribution of the two resistances to the total capacity of the pile depends on the density and shear strength of the soil, as well as on the characteristics of the pile material. The combined load-carrying factored capacity, Q_f, of a single pile is as given in the following geotechnical formula.

$$Q_f = Q_{tf} + Q_{sf} = A_t p'_L N_t + \Sigma f_s A_s$$

where:
Q_{tf} = total factored toe resistance
Q_{sf} = total factored shaft resistance
A_t = area of pile end
A_s = area of pile shaft per unit length of pile
p'_L = unit effective vertical soil stress at pile toe (below $Z = D_c$, use p'_{D_c})
N_t $\geqslant 2N_q$ (usually $= 2Nq$)
N_q = bearing capacity factor corresponding to the modified friction angle of the soil
L = total pile length in soil
z = depth co-ordinate
Σ = the summation over the pile length
f_s = unit factored shaft friction along the pile
f_s = $p'_z K_s M \tan\phi'_f \leqslant f_{D_c}$
p'_z = unit effective vertical soil stress at depth z
K_s = coefficient of earth pressure against the pile shaft
f_{Dc} = f_s for $z = D_c$
D_c = critical depth, the depth below which f_s and p'_L are constant
ϕ'_f = modified effective angle of shearing resistance (friction angle) in the soil
M = $\tan\delta'_f / \tan\phi'_f$
δ'_f = modified effective angle of wall friction for the pile-soil interface.

The ultimate bearing capacity of a pile in granular soil of constant density increases in a linear manner with increase in effective overburden pressure, at least down to a certain depth called the critical depth. Investigations indicate that there is only little increase in the end-bearing resistance and unit shaft resistance below the critical depth.

The ratio of the critical depth to the pile diameter generally increases with increases in the angle of shearing resistance and in the density of the soil. For most applications, the ratio ranges from a value of about e 10 at $\phi'_f = 25°$ to a value of about 20 at $\phi'_f = 35°$. However, since the density of most granular soils increases with depth, and since many other factors influence the validity of the theoretical estimates of pile capacity, a study should be made of Reference 40 before finalizing the design.

The value of M varies from 0.7 to 1.0, depending on the pile material (steel, concrete, wood) and method of installation.

In estimating the factored unit shaft resistance for piles shorter than 15 m, in particular for piles in clay, the product $K_s M \tan \phi'_f$ can be approximated to the value of 0.25 so that:

$$f_s = 0.25p'_z$$

The above effective stress methods of calculation for f_s are applied generally for piles in both granular and cohesive soils. However, in clay, a total stress method is sometimes used,

where: $Q_{tf} = 9A_t c_f$
$Q_{sf} = \Sigma c_f msrA_s$

where: c_f = factored undrained shear strength (i.e., factored cohesion)
Σ = summation over the pile length
m = adhesion coefficient
s = shape coefficient
r = regeneration coefficient

The adhesion coefficient, m, depends on the pile material and
m = 1.0 for piles of concrete and wood;
m = 0.7 for piles of steel.

The shape factor, s, is
s = 1.0 for cylindrical or prismatic piles, and
s = 1.2 for tapered piles.

The regeneration coefficient, r, is based primarily on the original shear strength of the material, and
r = 1.0 for soft clay,
r = 0.7 for stiff clay, and
r = 0.4 for hard cohesive glacial tills.

References 18, 40, and 51 provide additional information.

The geotechnical capacity of a pile is a function of the particular hammer-capblock-cushion-pile-soil system. The mobilized capacity and the developed driving stresses in the pile depend on this combination system and, in particular, on the elastic modulus of the pile material, whereas the strength of the pile material has only little influence.

In driven steel piles, dynamic measurements have verified that the developed driving stresses are about 200 MPa and do not exceed 250 MPa. consequently, although an improved yield strength increases the pile structural capacity, it does not increase the pile geotechnical capacity. Instead, the geotechnical capacity of a driven deep foundation unit at the time of the driving is governed by the dynamic impedance of the unit cross-section, EA_p/C. The geotechnical capacity usually increases after driving due to soil set-up. On the other hand, the capacity can also be reduced due to soil relaxation. Therefore, in the absence of field verification of the existence and magnitude of soil set-up, the geotechnical factored capacity of a unit is limited to the value of $2.5EA_p/C$. This limit applies to driven steel, concrete, and wood units.

The value of 2.5 E/C for steel units is relatively constant and equal to 100 MPa. For precast concrete units — ordinary reinforced, as well as prestressed — both the elastic modulus and the strain-wave speed vary and the value of 2.5E/C is not a constant. However, it is usually within the relatively narrow range of 15 to 18 MPa, and 15 MPa is recommended for use in design.

The dynamic impedance of a closed-end steel pipe pile can be substantially improved by concreting the pile before restriking. The resulting increased dynamic impedance (new combined value of $2.5\ EA_p/C$) will enable the unit to be driven to a higher geotechnical capacity and/or verify the existence of soil set-up.

Soil set-up can be verified in the field by test loading, or dynamic measurements. While restriking is a highly recommended method of quality control and will indicate soil relaxation, it does not provide sufficiently reliable information on soil set-up unless the pile impedance is increased or a heavier hammer is used that can develop more force and driving energy per blow.

The long-term structural capacity of wood piles reduces with time and can become smaller than the geotechnical capacity at the time of driving. The reasons for the strength loss are loss due to long-term loading, leaching, chemical aggression, anaerobic bacteria corrosion, and conditioning for preservative treatment and the treatment itself. Therefore, a conservative approach is recommended to the routine determination of long-term loading of wood piles. References 7 and 22 are recommended for more detailed study.

It should be recognized that because of soil set-up and other effects, the actually available capacities can often be considerably higher than the limit given by Clause 6—8.3.3.2. Therefore, when considered economical and suitable, the capacities should be determined according to Clause 6—8.3.3.1 instead.

C6 — 8.3.4 Group Effect

C6 — 8.3.4.1 In granular soils, provided the spacing is as given in the Code, the total bearing capacity of a pile group may be taken as the sum of the bearing capacities of the individual piles in the group.

For piles driven in clays having an undrained shear strength that is smaller than 100 kPa, and where the pile spacing is greater than 4 times the average pile diameter, the total bearing capacity of a pile group should be taken as 0.7 times the sum of the bearing capacity of the individual piles. Additional reduction of group capacity in clays must be considered where the pile spacing is smaller than 4 times the average pile diameter.

C6 — 8.3.4.2 For deep foundation units subjected to eccentric loading, the uniform distribution of the compressive loads in the units over the effective area A'_c, and the uniform distribution of the uplift loads in the units over the effective area A'_t of the group should be used for the geotechnical analysis of the group at the ultimate limit states. For details see References 38, 41a and 41b.

For an eccentricity, e, of the resultant load, P, on the base of a cap of the group of width, B, the effective width, B', of the compression zone can be determined from the following equilibrium equations for vertical loads and moments, respectively. The equations have been derived by taking moments about the outer compressive edge of the cap.

$$V = nQ_f - n'Q_f'$$

$$V(B/2 - e) = nQ_f\,B'/2 - n'Q_f'(B + B')/2$$

where: $$B' = \frac{nB}{n + n'}$$

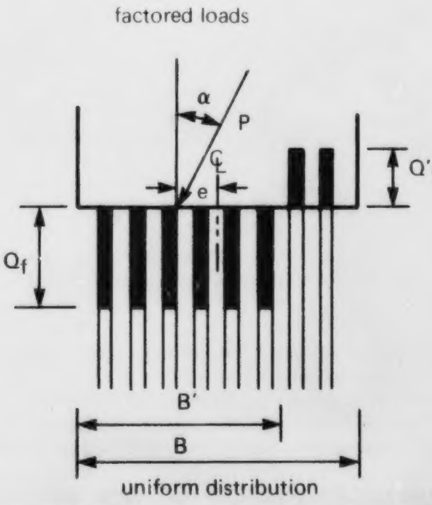
n = number of compression units of factored capacity Q_f

n' = number of tension units of factored capacity Q_f'

V = vertical component of resultant load P_u

For batter piles in a group, the vertical components of the factored capacities should be used in the above equations.

Figure **Ultimate Limit States Bearing Capacity for Geotechnical Analysis** **C6 — 8.3.4.2**



The above-mentioned method is on the safe side because it ignores the moment of the soil resistance from the cap to the point of the deep foundation units as well as the bending resistance of the units.

Figure C6—8.3.4.2 illustrates the uniform load distribution to A'_c and A'_t for a group of deep foundation units under eccentric inclined loading.

C6—8.3.4.3 This clause applies to deep foundation units where piles behave primarily as rigid units, see References 38, 41a and 41b.

C6—8.3.4.4 This clause deals with the determination of the reduction factor, R_i, by the use of Figure 6—8.3.4.3, where there are battered deep-foundation units.

To determine the inclination of the resultant load for determining the reduction factor, R_i, in the case of batter units, the effective angle of inclination, α', is given by $\alpha' = \alpha - \beta'$ as shown in Figure C6—8.3.4.4 where α is the inclination of the resultant load to the vertical and β' is the effective angle of inclination of the units group capacity to the vertical.

For details of group units under eccentric and inclined loads see References 38, 41a and 41b.

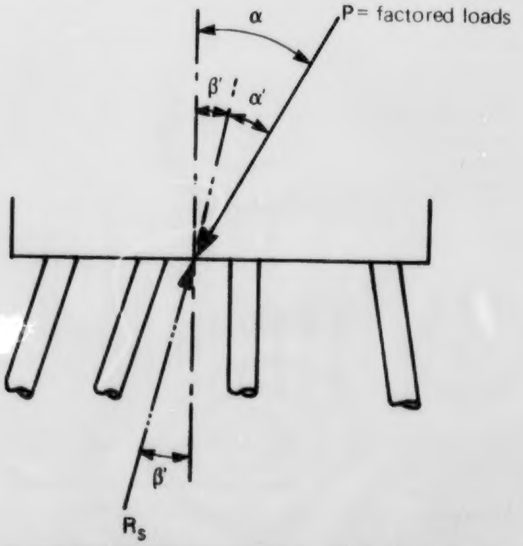
C6—8.3.4.5 In the structural design of a footing on piles for the ultimate limit states, the designer must consider two cases of load distribution on the piles. The first case corresponds to the distribution of the loads over a uniform compression distribution over the effective compression area, A'_c, and a uniform tension distribution over the effective tension area, A_t, as prescribed in Clause 6—8.3.4.2. In the second case to be considered, the pile loads are distributed linearly. Figure C6—8.3.4.5 illustrates the two cases of pile load distribution to be considered in the structural design of the footing.

Although uniform load distribution (per Clause 6—8.3.4.2) is considered adequate for the geotechnical check at the ultimate limit states, it is possible that some critical responses may occur in the footing itself prior to the development of the uniform compression and tension configuration assumed for the ultimate limit states. The assumption of a uniform pressure distribution applies for a loading condition causing the piles to be loaded close to their capacity at ultimate limit states and if the applied loading is not high enough, then the load distribution to the piles may be somewhat closer to a linear distribution or some distribution in between uniform and linear. This could be particularly so in the case of shorter piles where their support is principally end bearing on unyielding soils and rock and the response to imposed loading is largely elastic.

When assuming linear distribution of the pile loading for the structural design of the footing at ultimate limit states, the maximum pile load is not limited to R_i times factored axial capacity, because the soil supporting the pile may be sufficiently competent to cause an elastic response to the imposed loading.

The requirement to consider two pile load distributions for structural design at the ultimate limit states is new in Edition 2 of this Code.

Figure **Determination of Effective Angle of Inclination, α'** **C6—8.3.4.4**



C6 — 8.3.4.6 For the geotechnical analysis of deep foundation units, and for the structural design of the footing on the units, a linear load distribution to the units should be assumed when considering serviceability limit state, Type II. When considering serviceability limit states, Type II, it is assumed that the units will respond elastically to the imposed loading for concentric, eccentric and inclined loading.

Figure C6 — 8.3.4.6 illustrates the linear load distribution to a group of deep foundation units for both geotechnical analysis of the units and the structural design of the footing at serviceability limit states, Type II.

C6 — 8.3.5 **Downdrag**

When a soil deposit, in which, or through which, piles have been installed is subject to consolidation and settles in relation to the piles, downdrag forces are induced on the piles, through negative skin friction. The induced downdrag loads tends to reduce the usable pile capacity.

Negative skin friction is a pile capacity problem only in the case of a true end-bearing pile on very dense or hard soil or rock where the pile capacity is generally controlled by the structural strength of the pile and where settlements of the pile are negligible. In all other cases of piles bearing in compressible soils, where the pile capacity is controlled by point resistance and shaft adhesion or friction, the problem of negative skin friction may be regarded as a settlement problem.

See Reference 25.

Field observations on instrumented piles have shown that the magnitude of negative skin friction is a function of the effective stress acting on the pile and may be computed in a way similar to the calculation of positive shaft resistance. When designing for negative skin friction, it should be considered that the drag loads do not combine with live loads (e.g., wind and traffic loads) and, therefore, only permanent loads need be included with the drag loads, provided that the live loads are smaller than the drag loads (see Reference 26).

Negative skin friction can be reduced by applying a thin coat of bitumen on the pile surface. Before including the bitumen coating in the design, References 26 and 27 should be consulted.

C6 — 8.3.6 Bouyancy can affect the installation as well as the design of deep foundations and should be considered in the selection of the type of unit to be used.

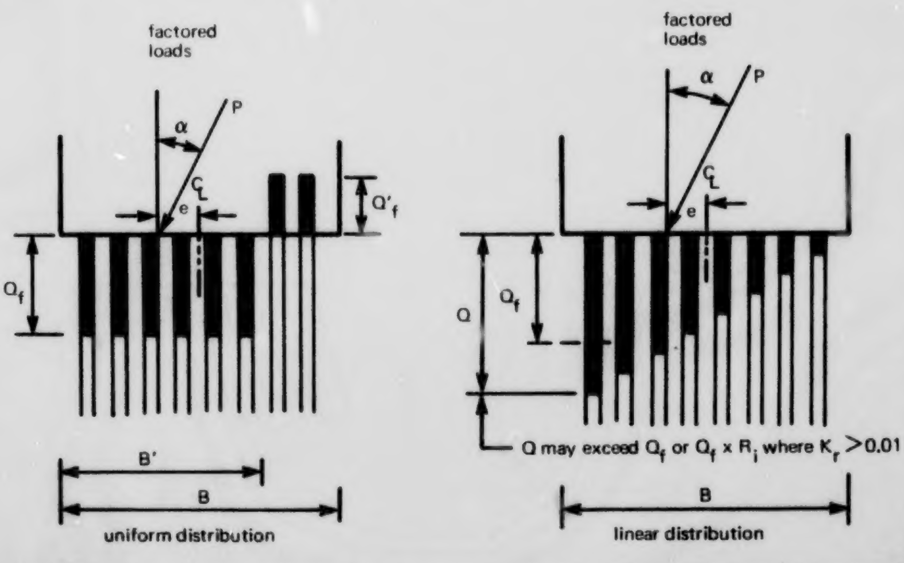
C6 — 8.3.7 **Uplift**

When deep foundations are subject to uplift forces, they should be investigated both for their resistance to pullout and their structural ability to resist tension.

C6 — 8.3.8 **Lateral Loads**

The horizontal load capacity of deep foundation units may be limited in different ways.

Figure **Uniform and Linear Load Distribution to Deep Foundation Units for the Structural Design of a Footing, Ultimate Limit States, Eccentric Inclined Loading.** **C6 — 8.3.4.5**



- For units which depend upon lateral soil resistance, the ultimate capacity of the soil may be exceeded — resulting in very large horizontal movements of the units and failure of the foundations.
- Excessive bending stresses in the units may cause them to fail.
- Deflections of the footings may be too large for the superstructure.

For piles subjected to lateral loads, the pile heads shall be fixed into the footing in accordance with Clauses 6—8.3.10.3 and 6—8.3.10.4. Any disturbed soil or voids created from the driving of the piles should be replaced with compacted granular material.

The methods currently available for the analysis of pile foundations subjected to horizontal loads are either empirical (and therefore approximate), or highly theoretical, requiring a sophisticated input of soil data, which usually involves a substantial uncertainty. Consequently, all methods must be used with due regards for their limitations. For a method of testing piles for horizontal loads, see References 12 and 18.

Most designers prefer to use empirical methods of analysis because none of the more complicated methods can predict the lateral capacity of the piles more accurately or reliably. It can be concluded from the satisfactory performance of the majority of pile structures that empirical methods are satisfactory for simple pile groups under average ground conditions, and lead to safe designs.

Normally, the lateral resistance of a pile group may be taken as the sum of the horizontal component of loads in the battered piles and the passive resistance of the soil surrounding the pile group. This assumes, however, that the battered piles can accommodate, by flexure or otherwise, the movement required to develop the full passive resistance. In cases where this assumption is doubtful (e.g., for short battered piles and/or where large movements are involved), the passive resistance should be neglected. References 12 and 18 provide additional information.

Where horizontal test loading data are not available, the horizontal passive resistance of a pile may be estimated by calculating passive earth pressure over an equivalent height equal to 6 times the pile diameter, and an equivalent width equal to 3 times the pile diameter. The pile diameter is the average diameter of round piles, or the average face-to-face distance of octagonal, hexagonal, and square piles within the equivalent height.

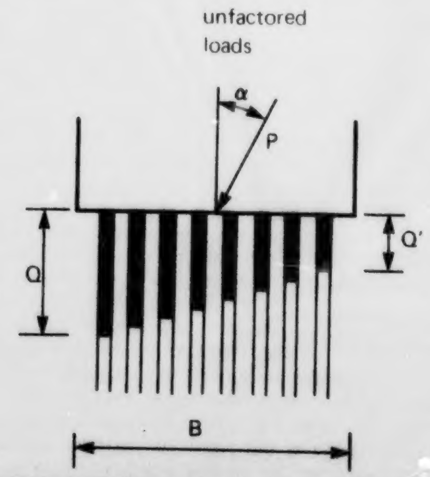
By an analogous approach, the lateral passive resistance of a pile group may be estimated by calculating passive resistance over an equivalent contact area consisting of an equivalent height equal to 6 times the average pile diameter and an equivalent length equal to:

$$L_e = L + 2B$$

where: L = the length, centre-to-centre, of the group in plan (perpendicular to direction of movement)

B = the width of the equivalent area of the group in plan (parallel to direction of movement).

Figure **Serviceability Limit States Bearing Capacity** **C6—8.3.4.6**



However, the lateral passive resistance of the pile group must not exceed the sum of the lateral passive resistance of the individual piles located on the equivalent length, L_e. See References 34 and 46.

C6 — 6.3.9 **Bearing Capacity at Serviceability Limit States**

Piles driven into granular deposits will undergo instantaneous settlement upon loading. However, for piles in cohesive soils, settlements are time-dependent and should be analysed by a geotechnical engineer to establish the magnitude and the period over which they are likely to occur. Additional information may be found in Reference 45.

C6 — 6.3.10 **Spacing Clearance and Embedments**

C6 — 6.3.10.2 This clause provides the minimum spacing of piles, in terms of the size and length of the pile, to ensure that adequate space is maintained between the piles to accommodate the driving equipment and to minimize the possibility of interference with adjacent piles. It has been shown that for long H-piles driven in clay, large deflections can occur about the weak axis, resulting in high bending stresses in the pile. It should be noted that non-parallel piles, or rows of piles with different inclination, can be spaced closer than parallel piles. Reference 30 provides additional information.

C6 — 6.3.10.3 The edge distance should be increased when the piles are subjected to large horizontal loads.

C6 — 6.3.11 **Structural Resistance**

C6 — 6.3.11.1 In the majority of cases, geotechnical rather than structural considerations govern the design of piles. Where piles extend above the groundline, such as in trestle-type piers, structural considerations can govern and the piles should be designed as columns.

C6 — 6.3.11.2 The lateral support provided by most soils has been found to be sufficient to prevent buckling. Exceptions are piles extending through water or soft mud.

It has been shown theoretically by Bjerrum [13] that buckling should be considered only if:

$$\frac{I}{A^3} < \frac{P^2 \max}{4cE}$$

where c is the coefficient of lateral displacement, equal to the horizontal subgrade reaction times the pile width, I is inertia, A is the area of cross-section, and E is the modulus of elasticity of the pile. Reference 14 provides additional information.

C6 — 6.3.11.3 Handling and driving seldom affect steel H-piles, steel tube piles, and wood piles. However, for reinforced concrete and prestressed concrete, handling and driving stresses are important considerations. References 3, 18, and 23 provide additional information.

C6 — 6.3.11.4 In the absence of certain chemically-aggressive compounds, corrosion of steel piling driven into undisturbed soil, regardless of the soil characteristics and properties, is usually insufficient to affect the strength or useful life of the piles. Such soil properties as type, drainage, resistivity, pH, or chemical composition are of no practical value in determining the corrosiveness of soils towards steel pilings driven underground [47].

The American Society of Civil Engineers' *Manual of Engineering Practice, No. 27* [8] on pile foundations and pile structures, states:

> "Steel piles entirely embedded in relatively impervious earth generally may be considered permanent. Entire embedment usually may be assumed as extending downward from a level about two feet below the ground surface. Atmosphere oxygen is blanketed off by the surrounding soil, thereby inhibiting progressive corrosion. Variations in groundwater level do not have any effect on the durability of steel piles under this condition, except as the upper layers are pervious to air and water."

The American Railway Engineering Association specifications [6] require steel piles to be protected from corrosion by concrete encasement or other suitable means. Protection of the piling at ground surfaces or normal waterlines, 300 mm above groundline or normal waterline and 1.0 m below the ground surface or low waterline, should be made by concrete encasement or other suitable means.

The amount of section reduction to be allowed for corrosion is difficult to predict because of the many variables involved. Where necessary to make an allowance for section reduction due to corrosion, the requirements given in References 1 and 20 provide additional information.

Under ordinary conditions, steel and concrete piles are durable. However, the presence of high concentrates of magnesium or sodium sulphate salts in the porewater can cause deterioration. Special precautions such as use of sulphate-resistant cement should be considered for concrete piles. References 14, 47, and 53 provide additional information.

C6 — 6.3.12 Driving Points or Shoes

Pile ends must be sound and undamaged after driving, because a pile with a damaged end:

- will drive to a capacity that is smaller than that of a sound pile,
- could drift and bend in the soil, or
- could twist and deviate from its intended location at the cut-off elevation.

Damage to the pile end can be minimized by means of a suitable pile end protection. For closed-end steel tube piles, a flat steel plate is normally sufficient protection. For steel H-piles, a separate shoe made of cast steel or ductile iron is needed. Methods, such as welding steel plates to the flanges and web of the pile, are rarely adequate. Concrete piles are usually provided with a steel plate cast into the pile and connected to the concrete by means of reinforcing dowels. Often, however, a regular rockshoe is required, which consists of a thick steel plate and a tip (dowel) made of hardened steel. Such rockshoes (rock points) are necessary for all piles driven to hard rock surface, where slippage of the pile end could occur.

The occurrence of pile end damage is frequent when driving in bouldery soils or into very dense soil layers. It is difficult to predict damage from borehole information and even to detect it from indirect observations during test driving or test loading. Severe tip damage can be discovered on wave traces from dynamic measurements. The best method to assess damage and to determine the necessity of pile end protection is to extract piles for direct visual inspection. Details of pile driving shoes are given in Reference 42. References 18 and 14 provide additional information.

C6 — 6.3.13 Splices

The design of a splice is a structural consideration. Splices should be designed to develop the full strength of the pile cross-section. The configuration of a splice should be such that it does not affect the performance of the pile, and that it becomes irrelevant where in the pile the splice is located.

i) **Splicing of Steel Piles**

Splicing of steel piles is commonly done by welding. Common practice is to make full strength butt welds using a back-up ring. Although this is not always necessary from a design view, it is good practice in order to accommodate the driving stresses and reduce potential bending. Pre-manufactured splicing sleeves can save considerable field time because half the welding can be done ahead of time.

ii) **Splicing of Precast Concrete Piles**

Quality requirements for concrete pile splices are stringent because of the determining influence of splices on the driveability of concrete piles. General requirements for mechanical splices are as follows.

- The strength of the splice must be comparable to that of the pile in compression, tension or bending.
- The splice must be designed and positioned so as to ensure and maintain the alignment of the jointed segments of a pile. Therefore, the splices must be cast square with the pile segment ends. Maximum permissible deviation (out-of-squareness) of the pile segment end is 1:100.
- The splice must be designed so that the tolerance play (slack) between two jointed segments of a pile is less than 0.5 mm in either compression or tension.

Inspection and Quality Control

C6 — 6.4

The design of deep foundations is based on information available on the geotechnical conditions at the site, and on the assumption that the deep foundation units will be properly installed and that they are sound. The objective of the inspection and quality control is to ensure that this is the case. Any deviation from the conditions assumed in the design must be brought to the attention of the designer for possible adjustment of the design and/or installation work.

Methods for the inspection work are to be adjusted to the deep foundation system used and the appropriate technical level in each case. Suitable information is available in References 18, 22, and 24.

Test loading and dynamic measurements, as outlined in Clauses C6—3.3.1(a) and (b) respectively, are methods suitable for quality control.

Inspection down a driven pile, when possible, provides a significant increase of confidence in the soundness of a pile with regard to pile damage and excessive bending. When driving long piles in any type of soil, or shorter piles through soil containing obstructions, the piles can bend, dog-leg, and even break without this being recognizable by usual inspection means after the driving. Closed-end pipe piles provide greater assurance of soundness than other piles, because of the possibility of inspection of the curvature and integrity of the pile given by the open pipe. By casting a centre tube in the precast concrete pile, access is provided for inspection down this pile also. See Reference 18.

Piers, Abutments, and Retaining Walls

C6 — 9

General

C6 — 9.1 In addition to the specific requirements of Section 1, the economic, construction, and aesthetic aspects should also be considered. Reference 4 provides additional information.

Foundations

C6 — 9.3

C6 — 9.3.1 This clause is concerned with the global stability of a substructure unit or the bridge as a whole. Instability of the underlying soil or rock mass can affect a substructure unit or units and the safety of the bridge.

C6 — 9.3.2 When a shallow foundation is placed on smooth or inclined bedrock, the bearing capacity check at the ultimate limit states does not ensure stability against sliding.

C6 — 9.3.3 The intent of this clause is to provide for intentional or natural removal of soil or rock adjacent to foundation units which could affect the substructure unit.

Backfill and Drainage

C6 — 9.6

C6 — 9.6.1 Free-drainage granular backfill within the specified wedge will generally result in minimum lateral earth pressure. For walls in cut, the slope of the excavation behind the wall may have to be flatter than 60° to maintain stability during construction.

C6 — 9.6.2 The spacing of weep holes and the diameter of drain piles depends on the permeability of the backfill and soil, and should be shown on the plans.

References 1, 4, 6 and 20 provide additional information for piers, abutments and retaining walls.

Bin-Type Retaining Walls

C6 — 10

General

C6 — 10.1 Although the strength of bin-walls, their components, and connections should conform to the requirements of other applicable sections, Clause 6 — 12 permits the design of manufactured, fabricated, patented and proprietary products, including components and connections, to be based on approved empirical methods.

Design

C6 — 10.3 A bin-wall is one in which a soil mass acts as a gravity wall, with the bin members serving to keep the soil mass intact. Bin-walls should therefore be designed as gravity walls to resist overturning and sliding forces imposed by the retained soil and other superimposed loads. The overall stability of the soil mass supporting the bin-wall could be investigated where a bin-wall is founded on deep strata of cohesive soils or soils that are weak in shear resistance. Bin-walls should be provided with a batter on inclination to improve stability. For steel bin-walls, a batter of 1 to 6 is commonly used. If walls are installed without a batter, a wider base is usually necessary to provide stability.

Bin-walls are more adaptable to settlement and differential movements than rigid concrete walls. Some types of bin-walls, such as those made of metal, adjust better to differential movements than others, because of greater flexibility inherent in the component members and in the nature of the joint design. Bin-walls having concrete members require a good foundation soil to limit differential movements and prevent structural distress.

Bin-walls erected in a corrosive environment require special treatment to resist corrosion. Steel bin-walls are normally galvanized. For excessively corrosive conditions, an additional asbestos-bonded coating can be applied by the manufacturer. Bin-walls can also be field coated with fibrated asphalt mastic.

Construction Requirements

C6 — 10.4

C6 — 10.4.1 The fill material within the bin-walls should be pervious and preferably crushed stone, gravel, or other coarse granular material. The material should be well graded from a maximum size of 10 mm down and should be placed and compacted in lifts not exceeding 200 mm in height.

C6 — 10.4.2 The load on the wall and the load on the foundation soil can be excessive if free-draining granular material is not provided behind the wall.

C6 — 10.4.3 The perforated pipe should be placed at sufficient depth to drain the backfill effectively.

C6 — 10.4.4 To obtain the pressure distribution required and to minimize distortion of the bins, it is necessary to prevent excessive load being transmitted through the corner posts. The normal practice is to provide a compressible cushion or about 200 mm of loose material under the base plates of the corner posts.

C6 — 10.4.5 Bin-walls are usually placed at a depth of 600 mm or more below the final grade at the face of the wall to improve the resistance against sliding. Where it is necessary to prevent movements due to frost heave, consideration should be given to placing the underside of the bin-walls below the level of frost penetration. References 5 and 44 provide additional information on bin-walls.

Sheet Walls

C6 — 11

General

C6 — 11.1

C6 — 11.1.1 and C6 — 11.1.2 The shear wall systems and materials covered by this clause are the common types of shoring walls used for the support of vertical faces of excavations during the construction of bridge foundations or for permanent bulkheads.

C6 — 11.1.3 This clause applies to both temporary and permanent installations.

C6 — 11.1.4 Unless scour protection is provided for in the design, the depth of embedment should be measured from the lowest depth of anticipated scour.

Loads

C6 — 11.2

C6 — 11.2.1 The effect of temporary loads such as construction equipment materials should be included in the design.

C6 — 11.2.2 Active and passive earth pressure due to soil and surcharge loads should be calculated in accordance with Clause 6 — 6.1.2.1.

Design

C6 — 11.3

C6 — 11.3.2 In some cases, global movements can occur independent of the type of sheet wall and constitute a stability problem.

C6 — 11.3.3 References 18 and 43 provide additional information on sheet walls.

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C7 Concrete Deck Slabs

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Scope **C7—1**

The types of deck slabs to which the provisions of Section 7 apply are listed in the Scope. In particular, voided slabs, waffle slabs, and slabs of unusual geometry are not included. The effects of haunches and skew are discussed in the commentary to Clauses 7—5 and 7—6.

A distinct feature of Section 7 is that it encourages the use of an empirical design method specified in Clause 7—7.4 which reduces the design of the deck slab to a prescription of the reinforcement, provided that the system satisfies certain criteria. It is the intent of this section to promote use of the fact that arching action develops in slabs in which appropriate confinement is provided.

The section also encourages the use of the yield line method of analysis.

Limit States **C7—4**

The limit states referred to in this clause are discussed in other sections of the Code. The important limit state of cracking is discussed in the commentary to Clause 7—7. It should be borne in mind that the local effects due to loads at the ultimate limit state must be considered in addition to global effects to ensure that local rupture of the slab does not occur.

Span Lengths **C7—5**

The provisions in this clause, adopted mainly from AASHTO Specifications [1], apply to slabs subjected to superimposed loads of unchanging sign. They are valid for slabs that are rectangular in plan, or for skew slabs in which the angle of skew does not exceed 20°. The effects of varying thickness should be considered in the idealization of the system for analysis. In the presence of haunches, the slab span may be approximated as shown in Figure C7—5, which has been adopted from the French Code [2].

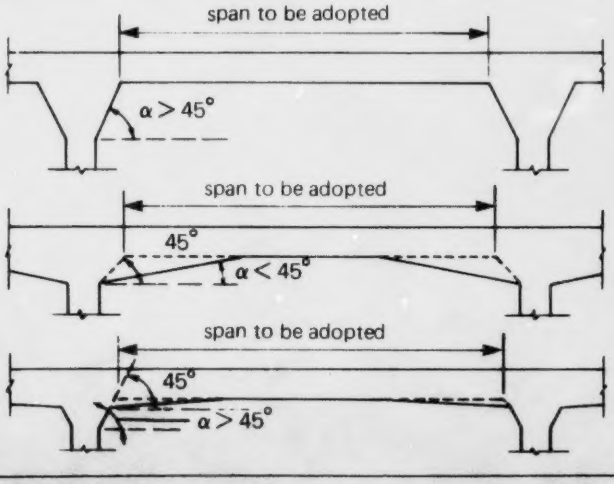
Methods of Analysis **C7—6**

General **C7—6.1**

Analysis and design of solid slabs should, where possible, be based on application of the yield line theory [3]. However, designs based on this method may lead to cracking which may be unacceptable at serviceability limit states. The range specified for the ratio of support to span moments makes it probable that excessive cracking will not result. This is adopted from the British Code [4]. Elastic methods of analysis, such as those due to Westergaard [5] or Pucher [6], may also be used to calculate the stress resultants in a slab. Refined elastic methods referred to in Section 3 are permissible, especially if they take into consideration in-plane stresses in the system.

A method for determining the moments in deck slabs due to concentrated loads is given in Clause 3—7.1.3.2. Alternatively, empirical formulae for the effective widths of slabs, as specified in the Canadian Standards Association [7] and AASHTO Specifications [1], which were developed mainly from Westergaard's method, may be used. However, this approach is conservative and results in unnecessarily high reinforcement requirements in typical bridge deck slabs in which arching effects are pronounced. Thus, a method incorporating considerations of arching effects [8] is preferred where it can be used.

Figure **Span Lengths for Haunched Slabs** **C7—5**



Some amount of edge restraint is required in a slab to enable development of arching action. The degree of edge restraint provided will vary with the configuration of the system and in some cases may be provided by the slab system itself, with or without edge stiffening.

Special Considerations **C7—6.2**

The degree of skew, corner conditions, haunches, etc., can have quite marked effects on the stress resultants in the slab, and should be considered in analysis. In the design of rectangular slabs, twisting moments are usually neglected and the slab is reinforced orthogonally for the longitudinal and transverse moments (i.e., in the assumed directions of the principal moments). In skew slabs, however, twisting moments can be high and the directions of the principal moments vary as a function of the angle of skew, the aspect ratio of the slab, and the pattern of loading [9]. The terms commonly used in describing skew slabs are defined in Figure 7—7.4.2. In general, the effect of skew can be neglected if the angle of skew does not exceed 20°, provided that the skew span is used to analyse the system. A simplified method for detailing deck slabs regardless of skew is presented in Clause 7—7.4, for those systems meeting the specific requirements. Otherwise, the slab should be analysed completely as a skew system and, where necessary, the effect of continuity should be considered [10]. It should be noted that the economical use of reinforcement in skew slabs will depend on the orientation of the reinforcement with respect to the skew span. The preferred orientation of reinforcement is shown in Figure 7—7.4.2.

Design **C7—7**

C7—7.1, C7—7.2, C7—7.3

Crack control is a very important requirement in deck slabs and must be considered as specified in Clause 7—7.2. The reinforcement cover requirements specified in Clause 8—9.3 are absolute minima and due consideration must be taken of normal construction tolerances in placing of reinforcement, since the amount of reinforcement cover has a very significant effect on deck durability. For slabs with an exposed concrete surface, the assumed effective depth for design purposes should allow for 10 mm of wear during the design life of the deck. Durable bridge decks can be achieved only through the provision of positive corrosion protection, use of high quality materials, careful attention to concrete mix proportions, and good construction practices.

Empirical Design of Composite Slabs **C7—7.4**

Clause 7—7.4 gives an empirical method for designing deck slabs which meet specified criteria. A minimum isotropic reinforcement ratio of 0.3% is specified for each face of the slab (i.e., four layers of reinforcement, each giving a reinforcement ratio of 0.3%). The method specified for determining effective depths for purposes of this clause allows the use of equally-spaced bars of the same diameter in each of the two layers of reinforcement in a face of the slab.

The requirement for a slab thickness of 225 mm represents Ministry practice with respect to minimum slab thickness. The empirical method is valid at lesser thickness, provided that span/thickness ratio does not exceed 15.

The elimination of intermediate diaphragms with reinforced and prestressed girders is a direct result of field tests conducted by the Ministry. No such testing has been done with steel I-beams and box girders without intermediate cross frames or diaphragms; hence, there is still a requirement for cross frames and diaphragms in the latter systems.

The provisions of Clause 7—7.4.2 are based mainly on recent studies which have revealed a high degree of arching action in bridge slabs [8]. The research findings have been confirmed by full-scale field studies conducted by the Ministry [11, 12, 13]. It was concluded from the several studies that if the conditions specified in Clause 7—7.4 are met then one can confidently expect the composite deck slabs to perform satisfactorily.

The requirement of 0.3% isotropic reinforcement applies to all deck slabs having skew angles not greater than 20°, and to the interior regions of a deck in which skew angles exceed 20°. However, in such a deck it is necessary to provide 0.6% isotropic reinforcement in the end portions as defined in Figure 7—7.4.2.

Horizontal rigidity must be provided in the system in order to resist the thrusts due to arching action. In the slabs studied, the necessary arching action could be ensured by the provision of cross frames or diaphragms at the spacings stated and by a cantilever slab about 1.0 m wide or an equivalent curb integral with the cantilever slab combination at the longitudinal edges of the bridge. All slab edges should be stiffened in accordance with Clause 7—8. The theory of arching action [8], indicates that the slab reinforcement index $\omega = \rho f_y/f'_c$ is an important parameter. A value of $f'_c \geq 31$ MPa was used in the studies.

In torsionally-stiff box girders connected only by the slab, it is possible for the overall transverse moment due to truck loading to become the criterion governing the reinforcement required in the slab. The 0.3% isotropic reinforcement which would result from the use of the empirical method may not be adequate for this purpose. In such cases, the clause includes a requirement for a detailed analysis to ensure that this possibility is taken into consideration. Longitudinal slab reinforcement in negative moment regions of continuous girders must also be provided for separately.

Edge Stiffening **C7 — 8**

The provisions in this clause are slightly modified versions of AASHTO and CSA Specifications.

Distribution Reinforcement **C7 — 9**

This clause conforms basically to AASHTO Specifications. However, the provisions do not apply to slabs designed by the empirical method specified in Clause 7 — 7.4. It is likely that the requirements of Clause 7 — 9 will be met by all slabs designed by the yield line method.

Cantilever Slabs **C7 — 10**

Particular care must be taken when using the yield line method for designing cantilever slabs, especially with respect to the choice of mechanisms. The elastic method proposed by Westergaard [5] has been widely used for determining the effects of concentrated loads on cantilever slabs. The AASHTO Specifications reflected in most of the provisions of Clause 7 — 10 are convenient approximations based on Westergaard's method. A semi-empirical method for the elastic analysis of a cantilever slab, in which the length of the supported edge is large in comparison to the span length, has been presented in Reference 14. The total moment at a point due to multiple wheel loads is obtained by adding the effect of individual wheel loads. The use of this method is specified in Clause 7 — 10.1(b) for the portions of longitudinal cantilever slabs away from the vicinity of the free transverse edges. In addition, the reinforcement requirements in cantilever slabs are increased in the region of an unsupported edge to take into account the increased moments in this region. Special attention should always be given to the development of cantilever reinforcement into adjacent portions of the deck slab.

Appendix A — 3.1(i) allows for the beneficial effects of barrier walls to be taken into consideration in the analysis of cantilever deck slabs. The semi-empirical method presented in Reference 15 can be used for this purpose.

It should be noted that as specified in Clause 2 — 4.3.5.4, horizontal loads and vertical wheel loads should not be applied simultaneously.

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C8 Reinforced Concrete

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Scope **C8 — 1**

This section contains the general requirements for the design of reinforced concrete structures and their components, including members having an average effective prestress less than 1.50 MPa. The design of members with an average prestress of 1.50 MPa and over is covered in Section 9, while the design of deck slabs is treated in Section 7.

Notation **C8 — 3**

ACI notation has been used wherever possible [1].

Materials **C8 — 4**

Concrete **C8 — 4.1**

The specific strength of concrete has been limited to 55 MPa, as the rectangular stress block used to calculate the flexural resistance does not accurately predict the behaviour of concrete when concrete strength exceeds this value [2].

Reinforcing Steel **C8 — 4.2**

For the purpose of design, the yield stress of reinforcing steel has been limited to 400 MPa because the strain necessary to develop high yield stress can result in unacceptable cracking.

Grade 350 reinforcement will be acceptable only if it meets the bend test requirements of Grade 400 reinforcement.

Reinforcement which is to be welded, and the welding procedure to be used, should be indicated on the plans.

Tack welding is not recommended since it can seriously weaken a bar by creating a metallurgical notch effect. Tack welding can be performed safely only when the material and the welding procedures are carefully controlled.

General Considerations **C8 — 5**

General **C8 — 5.1**

Statically indeterminate structures and their components should be proportioned for the maximum load effects specified in Sections 2. Elastic analysis methods should be used. The redistribution of negative moments in continuous members is not allowed.

Stiffness Assumptions **C8 — 5.3**

For statically indeterminate structures, the moment of inertia to be used in obtaining the relative stiffness of various members may be determined from either the uncracked concrete cross-section neglecting the reinforcement, or from the transformed cracked sections, provided that the same method is used throughout the analysis.

Material Properties **C8 — 5.4**

The values of moduli of elasticity for concrete and steel are those given in the ACI Standard 318-77 [3]. For normal density concrete with $w = 2300$ kg/m^3, E_c may be taken as $4800\sqrt{f'_c}$.

Flexural Tensile Stress at Cracking **C8 — 5.5**

This term has been adopted instead of the usual term "modulus of rupture".

Limit States **C8 — 6**

Serviceability Limit States **C8 — 6.2**

C8 — 6.2.1.2

Fatigue

The provisions for fatigue in concrete are based on Reference 4.

Table **Minimum Thickness for Components of Constant Thickness** **C8 — 6.2.2.3**

Superstructure Types	Minimum Thickness, Continuous (mm)	Minimum Thickness, Simple (mm)
Slabs	$\left[\frac{S + 3000}{30}\right]$ But not less than 225	$\left[\frac{S + 3000}{30}\right] \times 1.10$ But not less than 225
T-Girders	$\left[\frac{S + 2750}{18}\right]$	$\left[\frac{S + 2750}{18}\right] \times 1.10$
Box Girders	$\left[\frac{S + 3000}{20}\right]$	$\left[\frac{S + 3000}{20}\right] \times 1.10$

C8 — 6.2.2.2 **Cracking**

This clause is technically the same as that given in Reference 4; however, it is re-arranged to clarify the intent of the original expression given in Reference 11. There is a small possibility that the actual crack width could exceed the calculated value.

C8 — 6.2.2.3 **Deformations**

Excessive deformations may be avoided by using members whose thicknesses satisfy the minimum thickness requirements of Table C8 — 6.2.2.3. The values should be adjusted for variable thickness components.

When the thickness is less than that recommended in Table C8 — 6.2.2.3, deflections may be controlled by providing compressive reinforcement [5].

Performance Factors **C8 — 6.4**

The performance factors are calculated using procedure described in Reference 6.

Proportioning of Sections for Axial Load and Flexure **C8 — 7**

Assumptions at Ultimate Limit States **C8 — 7.2**

The provisions of this clause are basically the same as those given in Reference 3. The variation of strain across the depth of the section is assumed to be linear with the maximum strain at the extreme compression fibre equal to 0.003. The strain distribution and stress at ultimate condition are shown in Figure C8 — 7.2.

Assumptions at Serviceability Limit States **C8 — 7.3**

The provisions of this clause are based on Reference 7. Strain and stress are directly proportional to the distance from the neutral axis, except for deep flexure members. For members with overall depth-to-span ratios greater than two-fifths for continuous spans, and four-fifths for simple spans, a non-linear distribution of stress should be considered [8,9].

Flexural Members **C8 — 7.4**

C8 — 7.4.1 **Reinforcing Steel Limitations**

The tension reinforcement has been limited to $0.75\rho_b$ to ensure ductile behaviour at the ultimate limit states.

The minimum reinforcement requirements of Clauses 8 — 7.4.1.2 and 8 — 7.4.1.3 apply to members which, for reasons other than structural requirements, are much larger in cross-section than those required by strength considerations.

C8 — 7.4.2 **Factored Resistance**

The derivation of equations to compute factored resistance of typical sections are based on the assumptions stated in Clause 8 — 7.2.

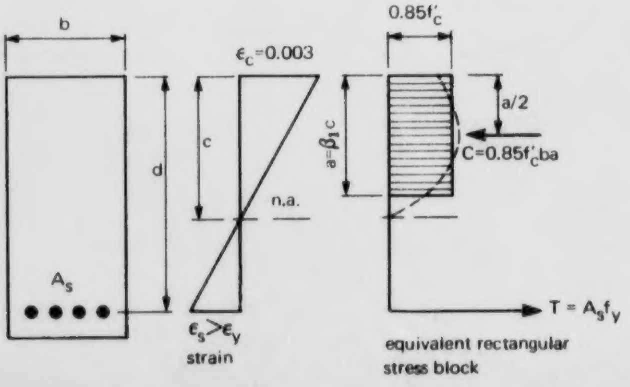
Compression Members **C8 — 7.5**

C8 — 7.5.1 **General**

The provisions of this clause are based on Reference 3. In computing the maximum eccentricity, movement of the superstructure due to shrinkage and temperature should also be taken into consideration.

The following expressions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

Figure **Assumptions at Ultimate Limit States** **C8 — 7.2**



1/ Pure Compression

The factored axial compression at zero eccentricity P_o may be computed by:

$$P_o = \phi\,[0.85\,f'_c\,(A_g - A_{st}) + A_{st}\,f_y]$$

For design, the pure compression factored resistance is a hypothetical loading condition since Clause 8 — 7.5.1.3 limits the factored axial resistance of compression members to 85% or 80% of the factored axial compression resistance at zero eccentricity.

2/ Pure Flexure

The assumptions given in Clause 8 — 7.2 or the applicable equations for flexure given in Clause 8 — 7.4 may be used to compute the factored resistance in pure flexure.

3/ Combined Flexure and Axial Load

The factored resistance for members under combined flexure and axial load should be based on stress and strain compatibility using the assumptions given in Clause 8 — 7.2. The factored resistance of a cross-section is controlled by yield in the tension reinforcement when the applied factored load P is less than P_b. The factored resistance of a cross-section is controlled by compression when the factored axial load P is greater than P_b.

4/ Balanced Strain Conditions

Balanced strain conditions for a cross-section are defined in Clause 8 — 2. For rectangular section with reinforcement in one or two faces located at approximately the same distance from the axis of bending, the factored resistance P_b and M_b may be computed by:

$$P_b = \phi\,[0.85\,f'_c\,ba_b + A'_s f'_s + A_s f_y]$$

and

$$M_b = P_b e_b$$

$$= \phi\left[0.85\,f'_c ba_b\left(d - d'' - \frac{a_b}{2}\right) + A'_s f'_s\,(a - d' - d'') + A_s f_y d''\right]$$

where:

$$a_b = \left[\frac{600}{600 + f_y}\right]\beta_1 d$$

and

$$f'_s = 600\left[1 - \left(\frac{d'}{d}\right)\left(\frac{600 + f_y}{600}\right)\right] \leq f_y$$

C8 — 7.5.2 **Reinforcement Limitations**

The provisions of Clause 8 — 7.5.2.1 are to ensure ductile behaviour at the ultimate limit states.

The limitation of ρ_t to 0.08 is the same as prescribed in Reference 4. However, it is desirable to limit this value to 0.04. When lapped splices are used, ρ_t should be based on total area of longitudinal reinforcement in the cross-section. For members in which the cross-section area is larger than that required by analysis, the minimum value of $\rho_t = 0.01$ based on the actual cross-section can result in unnecessarily large amounts of reinforcement. In such a case, the value of A_g should be based on the cross-section required by analysis.

C8 — 7.5.3 **Biaxial Loading**

The provisions in this clause are the same as given in Reference 4. An interaction surface diagram such as the Bresler Reciprocal Method or the Bresler Load Contour Method may be used in the investigation of a section [5, 10].

C8 — 7.5.4 **Slenderness Effects in Compression Members**

The slenderness of compression members determines the degree to which second order deformations affect the forces and moments acting on critical sections and the stability of individual members or groups of members. In general, the term "slender" is used to denote compression members in which the effects of second order deformations are signicant.

The design of a compression member by the use of the concept of magnified moment procedure involves the following stages.

a) A first-order structural analysis is carried out to determine the forces and moments.
b) The member is checked to determine if $k\ell_u/r$ is less than the value specified in Clause 8 — 7.5.4.2(d) and (e).
c) The maximum moment acting on the member is multiplied by a moment magnifier which accounts for second order effects.
d) The cross-section is proportioned to resist the magnified moment and the axial force from the first-order analysis.

A column is considered as braced if the bracing elements resisting lateral movement have a stiffness at least six times that of the column at the same level.

In bridge structures, supports such as pier bents and cantilever columns are generally unbraced. Some typical examples of ℓ_u are shown in Figure C8—7.5.4(a).

The value of k varies significantly. Depending on the degree of rotational and lateral restraints, k may vary from 0.5 to 1.0, while for columns which are free to sway laterally, k may be greater than 2.0.

Effective length factors for columns with various end conditions are shown in Figure C8—7.5.4(b). Approximate values of k for columns of constant cross-section in multi-bay frames may be obtained graphically from Jackson and Moreland Alignment Charts [11].

When a compression member is subject to bending in both principal directions, the effects of slenderness should be considered in each direction independently.

Since the restraints and section properties are different in each direction, it may be necessary to include slenderness effects in one direction and not in the other direction.

Bearing **C8 — 7.6**

The provisions of this clause are based on Reference 4. For bearings under high edge stresses or stress concentration, the bearing area should be designed in accordance with Clause 9—17.1. The details of reinforcement should be similar to those used in the anchorage zones of post-tensioned members.

Deformations **C8 — 7.7**

The provisions in this clause are based on Reference 3. For continuous spans, the effective moment of inertia I_e may be taken as the average of the values at the critical positive and negative moment sections.

Figure Unsupported Length (ℓ_u) **C8 — 7.5.4(a)**

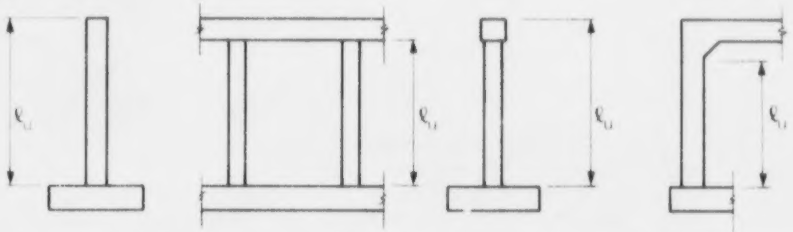
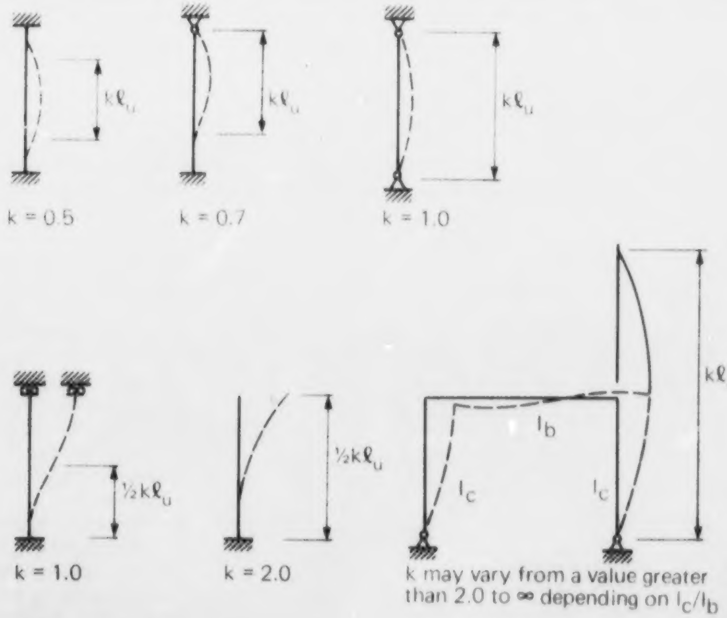


Figure **Theoretical Values of the Effective Length Factor** **C8 — 7.5.4(b)**



I = moment of inertia

Shear and Torsion

C8 — 8

Shear

C8 — 8.2

This clause is based mainly on the shear provisions of the ACI Standard 318-77 [3] and the suggested revisions thereto, as proposed in the report of ACI-ASCE Committee 426 [12].

C8 — 8.2.3

A minimum area of shear reinforcement is specified so as to restrain the growth of inclined cracks which may occur because of unexpected tensile stresses. Such reinforcement should increase the ductility of the member and, hence, provide a warning in situations where in an unreinforced web the sudden formation of inclined cracking might lead directly to distress. Slabs and footings are exempted from the minimum shear requirement because it is assumed that their ability to redistribute the load in the transverse direction will provide sufficient ductility. Minimum shear reinforcement is also not required if it can be shown that the applied shear is only a small fraction of the shear required to cause inclined cracking. This exemption, however, should be applied with caution, particularly in the case of members of major structural importance, because of the difficulties involved in accurately assessing all of the factors which possibly contribute to the formation of an inclinded crack.

C8 — 8.2.4

The expression in Clause 8 — 8.2.4 assumes that the shear strength of a reinforced concrete member is the sum of the shear resistances provided by two different sets of resisting mechanisms. Mechanisms of the first set V_c rely on the tensile strength of the concrete and their contribution is assumed, for typical sections, to be equal to the shear required to cause inclined cracking. Mechanisms of the second set V_s rely on the tensile strength of the shear reinforcement and the compressive strength of the cracked concrete. Their contribution is assumed to be given by the equations of the traditional truss analogy where the compression diagonals are assumed to be inclined at an angle of approximately 45°.

In a member of variable depth, the components of the "flange forces", namely the tension and compression forces resisting the moment parallel to the shear, will affect the shear resistance of the member.

If the member depth increases in the direction of increasing moments, the flange force components can be added to the shear resistance of the member. If the member depth decreases in the direction of increasing moments, the flange force components should be subtracted from the shear resistance of the member.

C8 — 8.2.5

Shear resistance near a concentrated load or reaction is increased if compression is introduced into the member, for example, when a downward load is applied to the top face and an upwards reaction is applied to the bottom face. Hence, sections located at a distance less than d from the supports of such a member need not contain more shear reinforcement than that provided at a section located at distance d from the face of the support. Under some reaction conditions, such as a member supported by cross members framing into its sides, compression will not be introduced into the member and the shear resistance must then be checked at the face of the support.

C8 — 8.2.7

The basic expression for V_c given in Clause 8 — 8.2.6 applies to members having a/d ratios greater than 2 and is controlled by the tensile strength of the concrete.

The basic shear stress v_b provided by the concrete is a function of the amount of longitudinal reinforcement in the section [12, 13]. In calculating ρ_w, at a section, only that reinforcement which is adequately anchored should be included.

C8 — 8.2.8

Special treatment is given to one-way slabs in Clause 8 — 8.2.8 because test data show that v_b is insensitive to ρ_w for such members [12].

C8 — 8.2.9

Axial compression will increase the shear resistance of members. The expression given in Clause 8 — 8.2.9 accounts for this increase and is approximately the lower-bound of the test data on axially-compressed beams [12].

C8 — 8.2.10

Axial tension reduces the shear resistance of members. If the tension is not considered when determining the required area of longitudinal reinforcement, V_c must be taken as zero. If the tension is taken into account, then V_c may be taken as varying between $v_b b_w d$ and zero as the axial tensile stress varies between zero and $0.50\lambda\sqrt{f'_c}$.

C8 — 8.2.11 and C8 — 8.2.12

These clauses give the traditional truss analogy expressions for the shear resistance V_s, which depends on the tensile strength of the shear reinforcement. Because of the inferior crack-control characteristics of bent bars, upper limits are placed on V_s when such bars are used.

C8 — 8.2.13 For high intensities of shear loading, close spacing of the shear reinforcement should be provided in order to control the growth of inclined cracks.

C8 — 8.2.14 The limiting of V_s to $0.66\ b_w d\lambda\sqrt{f'_c}$ ensures that the growth of inclined cracks is adequately controlled, while limiting the sum of $(V_c + V_s)$ to $0.20\ b_w df'_c$ ensures that the concrete in the web does not crush under the diagonal compression. Both of these limits have been derived primarily for the case of shear reinforcement perpendicular to the longitudinal axis. Because inclined stirrups provide better crack control, and result in lower diagonal compressive stresses, these limits will be more conservative for this type of reinforcement.

C8 — 8.2.15 When a girder supports a beam which frames monolithically into the girder, the joint region between the beam and the girder requires special "transmission" reinforcement. The function of this reinforcement is to carry the beam load from the bottom of the girder where it may have been deposited by the diagonal compression struts of the beam, up to compression zone at the top of the girder as shown in Figure C8—8.2.15. This transmission reinforcement should preferably be composed of stirrups surrounding the main reinforcement of the support girder and located within or immediately adjacent to the joint area. Where practical, the main reinforcement of the supported beam should be placed above the main reinforcement of the supporting girder.

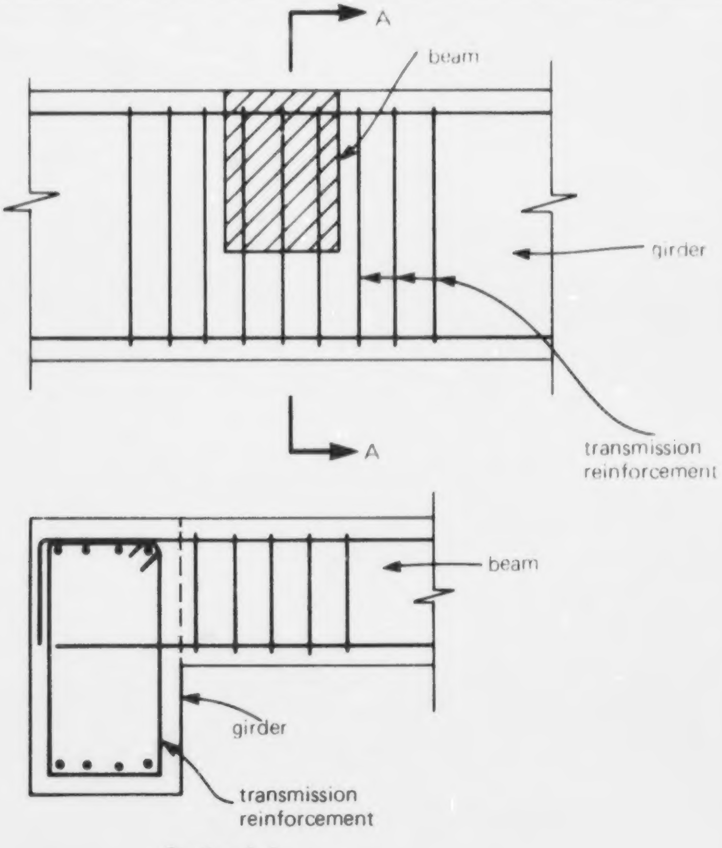
Torsion

C8 — 8.3 For the purpose of this clause, the magnitude of the applied torque should be determined by an elastic analysis in which uncracked stiffness values are employed.

This clause is based primarily on the torsion provision of CSA Standard A23.3 [14].

C8 — 8.3.1 A torque corresponding to $\phi\ 0.10\ A_c t_c \sqrt{f'_c}$ is about one-quarter of the torque required to crack the member. Torques of such a small magnitude will not cause a significant reduction in either flexural or shear strength.

Figure **Transmission Reinforcement** **C8 — 8.2.15**



Section A-A

C8 — 8.3.3 In design it is appropriate to distinguish between two different types of torsional loading: *equilibrium torsion*, where the torque is necessary to maintain equilibrium in the structure; and *compatibility torsion*, where the torque arises only because of the restraint to angular rotation induced by adjacent members.

If the torsion is required by equilibrium, then the member must be designed to provide the required torsional strength. In the case of compatibility torsion, the member must be designed so that it is capable of undergoing the required torsional deformation.

To classify the type of torsional loading, it is helpful to assume that the member has zero torsional stiffness. If the structure is unstable when the torsional stiffness is zero, then the torsion is required by equilibrium.

After torsional cracks form, a properly detailed reinforced concrete member can undergo a substantial twist with only a small increase in the torsion. Hence, in cases where the torsion is not needed for equilibrium, it is a reasonable design approach to provide a minimum amount of torsional reinforcement and assume that the torsional stiffness, and hence the torsion, is zero. In reality, there will be some torsion in the member and the presence of this torsion should be allowed for when detailing adjacent members. Further, the presence of this small torsion will somewhat lower the shear required to first yield the web reinforcement.

C8 — 8.3.4 If the member contains insufficient torsional reinforcement, it will fail in a brittle manner upon the formation of the first torsional crack. If more than one torsional crack is to form, the post-cracking torsional resistance of the member should be equal to, or greater than the cracking torque. The expression in Clause 8 — 8.3.4 is based on this condition. In deriving the expression in Clause 8 — 8.3.4, it is assumed that the cracking torque is a function of A_c and t_c while the post-cracking torsional resistance is a function of A_o and p_o.

C8 — 8.3.5 and C8 — 8.3.6 These two clauses give the basic equations for determining the amount of torsional reinforcement required to provide a certain torsional resistance. These equations are based on the 45° space truss analogy [15].

The definitions for A_o and p_o are based on the convenient design assumption that the path of the shear flow passes through the centres of the longitudinal corner bars of the hoops. More accurate assessments of the dimensions of the shear flow path can be made if desired [16].

In satisfying the requirements of Clause 8 — 8.3.6, it is not necessary to place all of the longitudinal torsional reinforcement A_ℓ around the perimeter of the section provided that one bar of sufficient size is placed in each corner of the hoop. The remainder of the longitudinal reinforcement can be placed around the perimeter of the member.

C8 — 8.3.7 Torsion causes tension in the longitudinal reinforcement. In the compression zone of a flexural member the compression due to flexure will counteract the tension due to torsion and, hence, reduce the need for longitudinal torsion reinforcement in this portion of the section.

C8 — 8.3.8 This clause is intended to ensure that members which must display torsional strength do not suffer premature failure due to crushing of concrete in the web prior to yielding of the reinforcement. The limiting nominal shear stress of $0.22f_c'$ is a conservative approximation to the values predicted for different members by the diagonal compression field theory [16].

C8 — 8.3.9 and C8 — 8.3.10 These clauses give rules for determining the effective dimensions of flanged and hollow sections. The rule for t_o is based on the assumption that prior to failure the concrete cover will spall off to the centreline of the hoops.

C8 — 8.3.11 Clause 8 — 8.3.7 assumes that all of the torsional resistance is provided by a closed flow of shear around the perimeter of the section, usually referred to as St. Venant or "circulatory" torsion. For thin-walled, restrained, open cross-sections, the vast majority of the torque will be provided by shear stresses which arise because of restraint of longitudinal deformations, i.e., warping torsion [17].

Shear Friction

C8 — 8.4 This clause is based on the provisions of Reference 3 and the revisions suggested in Reference 12.

C8 — 8.4.1 This clause defines where the shear friction provisions may be applied. Essentially, the concept is appropriate in those instances where a direct shear failure involving shear slip across a critical plane is possible.

C8 — 8.4.2 The basic shear friction approach is to assume that a crack has occurred in an unfavourable location and then to provide reinforcement across this crack to resist the shear slip. It is assumed that as shear slip occurs along the crack, the irregularities of the crack cause the opposing faces to separate to such an extent that the reinforcement crossing the crack yields in tension. To balance this tension in the reinforcement, compressive stress develops between the concrete faces. The presence of these compressive stresses enable shear stresses along the crack to be resisted by friction.

C8 — 8.4.3 Shear friction reinforcement is most effective when it is well distributed across the critical shear plane. The requirements in this clause for a minimum number and minimum amount of shear friction reinforcement are based on tests carried out at the University of Washington [18, 19].

C8 — 8.4.4 If too much shear friction reinforcement is used, the concrete will crush before the reinforcement yields. The restrictions in this clause are intended to prevent this mode of failure.

C8 — 8.4.5 and C8 — 8.4.6 These clauses give the basic shear friction equations.

For reinforcement perpendicular to the shear plane, the assumed shear resistance is simply the yield force in the reinforcement multiplied by the coefficient of friction. For reinforcement inclined such that the assumed shear slip introduces tension in the reinforcement, the shear resistance has two components: one due to the shear resisted directly by the component of the tension in the reinforcement parallel to the shear plane, and the other due to the shear friction which is a function of the component of the tension in the reinforcement perpendicular to the shear plane.

C8 — 8.4.7 The values for apparent coefficients of friction given in this clause are taken from Reference 3.

C8 — 8.4.8 Applying direct tension across the shear plane reduces the shear resistance, while applying direct compression increases it.

C8 — 8.4.9 In order to be effective, the shear friction reinforcement should be well distributed and anchored in such a manner that it can develop its yield stress at the critical shear plane.

C8 — 8.4.10 This clause provides requirements for surface finishes so that the coefficients of friction given in Clause 8 — 8.4.7 can be developed.

When concrete is placed against previously hardened concrete, the interface should be roughened so that the depth of indentation is 5 mm and the spacing of the indentation is not greater than 15 mm.

Deep Shear Spans

C8 — 8.5 These provisions are based on Reference 12. A deep beam, with respect to shear, is one in which a significant portion of the shear force is transferred directly from the load to the support by compressive strut action. If such action exists, the shear resistance of the beam will be greater than that predicted by the procedures of Clause 8 — 8.2. In general, this condition only exists, if the beam is loaded on its top face and supported on its bottom face. If a deep beam is loaded and supported on the lower face, a compression thrust line cannot develop and, hence, such a beam has essentially the same shear resistance as is given in Clause 8 — 8.2 for a shallow beam.

C8 — 8.5.1 and C8 — 8.5.2 These clauses cover two types of shear spans.

a) Shear span supporting a "principal load" which acts with a/d less than 2 at a distance of 2d or less from the support. In such a shear span, the thrust line is assumed to be a straight line joining the load and the reaction.

b) Shear span not supporting a principal load. Such shear spans are assumed to act as if uniformly loaded. The length is defined in terms of the distance ℓ_s from the support to the point of zero shear. The provision in Clause 8 — 8.5.1(b) applies to deep shear spans and not to beams. A beam can have both types of deep shear spans or, in the case of a load applied near to one end, a beam can have one deep shear span and one shallow shear span.

C8 — 8.5.3 It is assumed that the shear resistance of a deep beam can be calculated as the sum of the shear provided by the concrete, the vertical stirrups, and the horizontal stirrups.

C8 — 8.5.4 The upper limit for the shear resistance provided by the reinforcement is calculated in accordance with Clause 8 — 8.2.14 for a shallow beam, while the upper limit of total shear resistance is calculated as for shear friction in accordance with Clause 8 — 8.4.4. It should be noted that in order to obtain these high shear resistances, considerable care should be taken in detailing the longitudinal tensile reinforcement. In a deep beam, this reinforcement acts like the tie in a tied arch and should be extended to the support where it should be adequately anchored. Detailing recommendations for deep shear spans are given in References 20 and 22.

C8 — 8.5.5 This clause gives expressions for the magnitudes of the three components of shear resistance. The expression in Clause 8 — 8.5.5(b) assumes that the shear provided by the concrete increases as the shear span decreases. It should be noted that the value of the shear for deep shear spans exceeds the shear required to cause diagonal cracking.

The expression in Clause 8 — 8.5.5(c) assumes that the shear provided by the vertical stirrups decreases with shear span. As the shear span decreases, the diagonal cracks become more steeply inclined and, hence, they will intersect fewer vertical stirrups.

The expression in Clause 8 — 8.5.5(d) assumes that as the shear span decreases, the shear resistance provided by the horizontal stirrups increases. For short shear spans, this reinforcement acts like shear friction reinforcement. The quantity A_{vh} is the horizontal reinforcement provided to resist shear, and does not include the main longitudinal tensile reinforcement.

C8 — 8.5.6 This clause gives the equivalent shear span length for spans not supporting a principal load.

C8 — 8.5.7 and C8 — 8.5.8 These clauses give the limits on minimum amounts and maximum spacing of shear reinforcement. These limits are intended to provide sufficient reinforcement to control possible shear cracking.

Brackets and Corbels

C8 — 8.6 The provisions in this clause are taken from Reference 21.

When subjected to loads, a bracket or corbel tends to act as a simple truss or a very short beam which can fail in a number of ways depending on its details and dimensions. Thus, a corbel may fail by shearing along the interface between the column and corbel, by yielding the tension tie, by crushing the compression strut, or by localized bearing or shearing failure under the loading plate.

Clause 8 — 8.6.2 specifies the forces to be considered in design, while Clause 8 — 8.6.3 requires that the amount of reinforcement needed to prevent shearing along the interface be calculated by the shear friction approach. The tension tie reinforcement area should be chosen by the procedure given in Clauses 8 — 8.6.3 to 8 — 8.6.6. These requirements are such that the reinforcement forming the tension tie yields at the ultimate limit states.

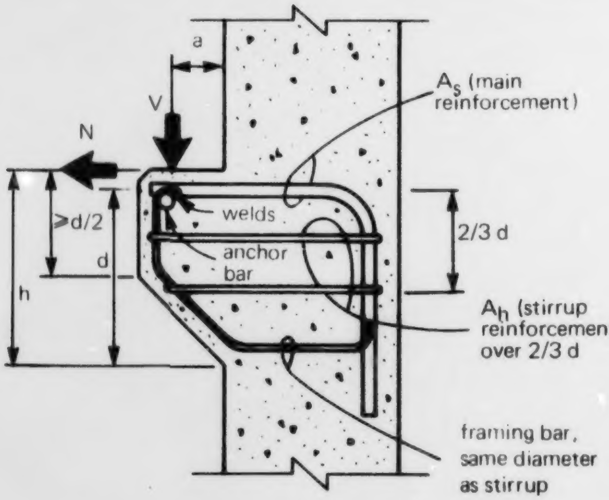
Further explanation of these provisions can be found in Reference 21. Figure C8 — 8.6 shows the typical reinforcement of a corbel.

Interface Shear For Composite Members

C8 — 8.7 The provisions in this clause are based mainly on the comparable provisions of Reference 4.

C8 — 8.7.3 The interface shear resistance of a composite member is checked by the provisions of these two clauses. The nominal shear stress on the interface is limited to a value which depends on whether the contact surfaces have been intentionally roughened or not. The nominal shear stress on the interface is calculated by dividing the total shear force acting on the composite section at the location being investigated, by the product of the depth of the composite section and the width of the interface. For high values of interface shear stress, reinforcement proportioned by the shear friction approach may have to be provided.

Figure **Typical Corbel Reinforcement** **C8 — 8.6**


a
V
N
A_s (main reinforcement)
welds
≥d/2
anchor bar
d
h
2/3 d
A_h (stirrup reinforcement) over 2/3 d
framing bar, same diameter as stirrup

C8 — 8.7.4 The possibility of relative displacement between the separate components of a composite member can be investigated by the shear friction approach as in Clause 8 — 8.4. In applying this approach, it is assumed that a crack has occurred along the interface of the separate components. This potential failure plane is assumed to extend from the section of maximum positive moment to the section at which moment equals zero (Figure C8 — 8.7.4). The shear force transmitted across this plane is determined by the flexural stresses at the maximum moment section. The resistance to relative displacement is maintained by the clamping force produced by the yielding of all reinforcement crossing the potential failure plane.

C8 — 8.7.6 The definition of intentional roughness has been given in the Commentary Clause C8 — 8.4.10.

Figure **Interface Shear** **C8 — 8.7.4**

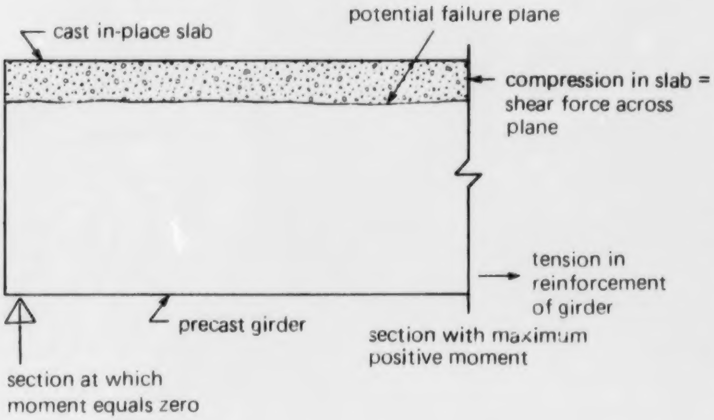
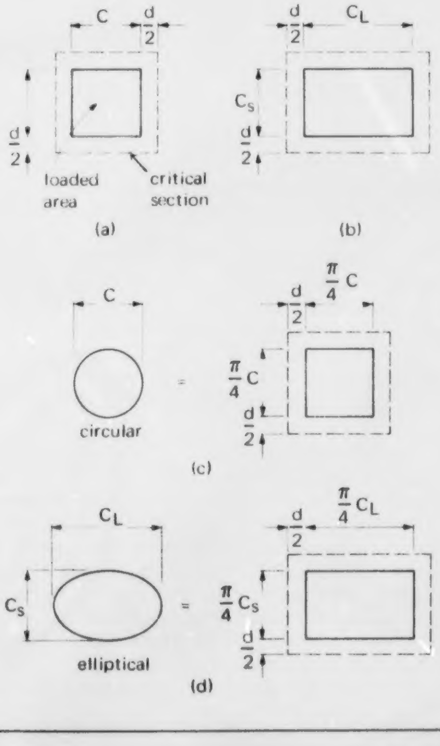

cast in-place slab
potential failure plane
compression in slab = shear force across plane
tension in reinforcement of girder
precast girder
section with maximum positive moment
section at which moment equals zero

Figure **Critical Sections** **C8 — 8.8.4**


C
d/2
loaded area
critical section
(a)
C_L
C_s
(b)
circular
π/4 C
(c)
elliptical
π/4 C_L
π/4 C_s
(d)

Slabs and Footings

C8 — 8.8 This clause is based on the shear provisions of References 3 and 12.

C8 — 8.8.2 In a slab without shear reinforcement, a diagonal tension crack forms at about half the collapse load at approximately one-half the slab depth from the periphery of the loaded area [12]. This diagonal crack is stable and seldom apparent on the surface of the slab. The shear resistance is taken as the shear at failure rather than the shear at inclined cracking. When shear reinforcement consisting of bent bars or stirrups is provided, such reinforcement becomes effective as soon as inclined cracks form.

C8 — 8.8.3 The shear resistance provided by the concrete is assumed to be a function of the shape of the loaded area. For square loaded areas, two-way action is effective and hence V_c is taken as $0.30\, b_o d \lambda \sqrt{f'_c}$. For very long, narrow areas, the slab on the long side will essentially be subjected to one-way action and hence V_c is taken as $0.15\, b_o d \lambda \sqrt{f'_c}$. For other than rectangular areas, β_c may be taken as the maximum overall dimension of the loaded area divided by the dimension measured at right angles to the direction of maximum overall dimension.

C8 — 8.8.4 This clause defines the critical section for determining b_o. The provision permitting the critical section to be parallel to the boundaries of the loaded area eliminates the theoretical need for circular segments adjacent to the corners of loaded areas. The b_o value of $(\pi C + 4d)$ permitted for circular loaded areas exceeds the $\pi(C + d)$ value at a critical section d/2 from the perimeter of the loaded area. This liberalization is in accordance with experimental observations [13]. Oval, elliptical, and other loaded areas of rounded shapes can be similarly transformed into rectangular shapes. The peripheral lengths of the round shape and of the equivalent rectangular shape should be the same; in addition, the ratio of the greater overall dimension of the loaded area to the dimension at right angles to that greater dimension should be the same. Critical sections adjacent to columns of different configuration are shown in Figure C8 — 8.8.4.

C8 — 8.8.6 This clause provides the upper limit for the shear strength of slabs with shear reinforcement. Typical examples are shown in Figures C8 — 8.8.7, C8 — 8.8.8 and C8 — 8.8.9.

Figure Placing of Shear Reinforcement **C8 — 8.8.7**

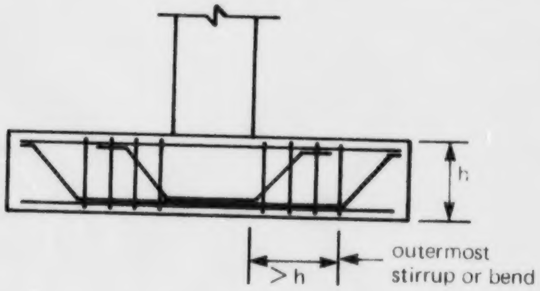
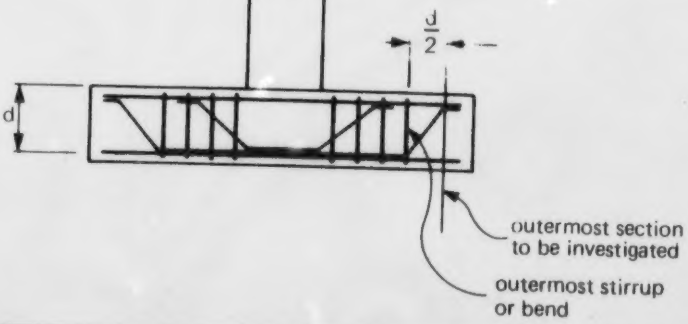
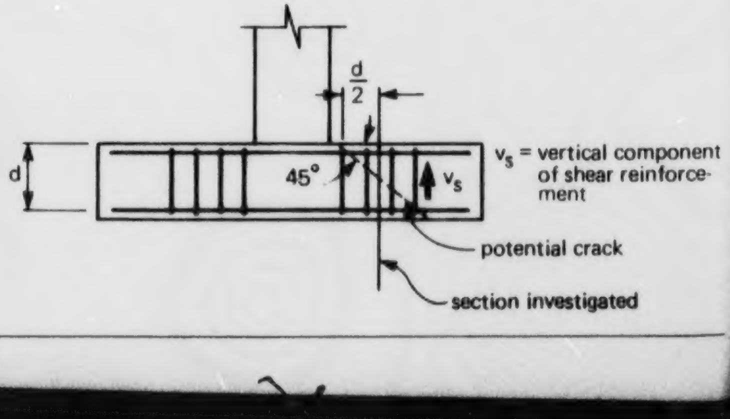


Figure Section to be Investigated **C8 — 8.8.8**


d/2
d
outermost section to be investigated
outermost stirrup or bend

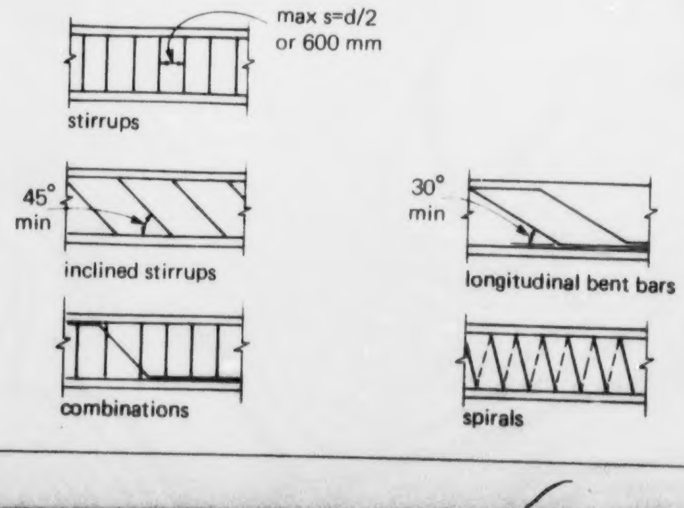
		C8 — 8.8.10	If the moment about the centroid of the critical section which must be transferred to or from the slab or footing exceeds 0.2 Vd, then account should be taken of the reduction in shear resistance which results from this moment transfer. In this regard, the provisions suggested in Reference 12 may be used.
	Alternative Methods of Proportioning for Bending, Shear and Torsion	**C8 — 8.9**	This clause enables the shear and torsional resistance to be checked by appropriately verified theoretical models rather than by the empirical procedures of Clauses 8 — 8.2, 8 — 8.3, 8 — 8.5, and 8 — 8.6. An example of such a model is the variable space truss described in References 22 and 23. Detailed design recommendations based on a development of this model are given in Reference 24. While the resistance of the member may be determined by theoretical procedures, the empirical limits on the minimum and maximum amounts of reinforcement should still be satisfied in order to ensure ductile behaviour of the member.
Details of Reinforcement		**C8 — 9**	
	Hooks and Bends	**C8 — 9.1**	
		C8 — 9.1.1 and **C8 — 9.1.2**	The provisions in these clauses are similar to those given in Reference 3. Where any bar requires unusual bends or where bars are field bent or partially embedded in concrete, the techniques used should be approved. The bending of reinforcement in sizes 45 and 55 should be avoided.
	Spacing of Reinforcement	**C8 — 9.2**	
		C8 — 9.2.1 to **C8 — 9.2.6**	The provisions of these clauses are similar to those of Reference 3. The spacing requirements in these clauses apply also to the clear distance between spliced bars. Bundled bars are permitted provided that they are located within stirrups or ties.
	Concrete Protection For Reinforcement	**C8 — 9.3**	
		C8 — 9.3.1	The minimum cover values are related to different types of concrete members and exposure conditions such as weather, earth, and de-icing chemicals. In corrosive or marine environments, or in unusually severe exposure conditions, the minimum concrete cover should be increased or other protection such as coated bars, waterproofing, or high-quality durable concrete provided. The tolerances in placing the reinforcement are based on experience.
		C8 — 9.3.2	Methods such as epoxy coating, galvanizing, or metallizing have been proven to be satisfactory.
	Distribution of Reinforcement in Flexural Members	**C8 — 9.4**	The provisions of this clause are similar to those in Reference 3. A proper distribution of reinforcement will minimize cracking. For deep short-span beams, the amount of steel for the side forces may have to be increased according to Reference 26.
	Shear Reinforcement	**C8 — 9.6**	The provisions in this clause are similar to those in Reference 3. Typical types of shear reinforcement are shown in Figure C8 — 9.6.1.

Figure **Resistance of Shear Reinforcement** **C8 — 8.8.9**



	Torsional Reinforcement	**C8 — 9.7**	The provisions of this clause are the same as those in Reference 25.
	Lateral Reinforcement for Compression Members	**C8 — 9.8**	The provisions of this clause are similar to those in Reference 3.
	Shrinkage and Temperature Reinforcement	**C8 — 9.9**	The provisions of this clause are similar to those in Reference 3.
Development and Splicing of Reinforcement		**C8 — 10**	
	Development Requirements	**C8 — 10.1**	The provisions of this clause are similar to those in Reference 3. When bars are cut off in the tension zone, the shear capacity and the ductility of the member are reduced. The application of the development length for the longitudinal reinforcement of a continuous beam is illustrated in Figure C8 — 10.1(b). The provisions of Clause 8 — 10.1.2.3 will limit bar size for positive moment reinforcement at the point of inflexion and at simple supports, in order to provide design restraint for flexural bond at points of small moments and large shears. Such a condition could exist in a heavily-loaded beam of short span in which large diameter bars are used, as illustrated in Figure C8 — 10.1(a).
	Development Length of Deformed Bars and Deformed Wire in Tension	**C8 — 10.2**	The provisions of this clause are the same as those in Reference 3.
	Development of Deformed Bars in Compression	**C8 — 10.3**	The provisions in this clause are the same as those in Reference 3.
	Development Length of Bundled Bars	**C8 — 10.4**	The provisions in this clause are the same as those in Reference 3.
	Standard Hooks	**C8 — 10.5**	The provisions in this clause are the same as those in Reference 3.
	Development of Welded Wire Fabric in Tension	**C8 — 10.7**	The provisions in this clause are the same as those in Reference 11.

igure **Typical Types of Shear Reinforcement** **C8 — 9.6.1**



Anchorage of Shear Reinforcement **C8 — 10.9**

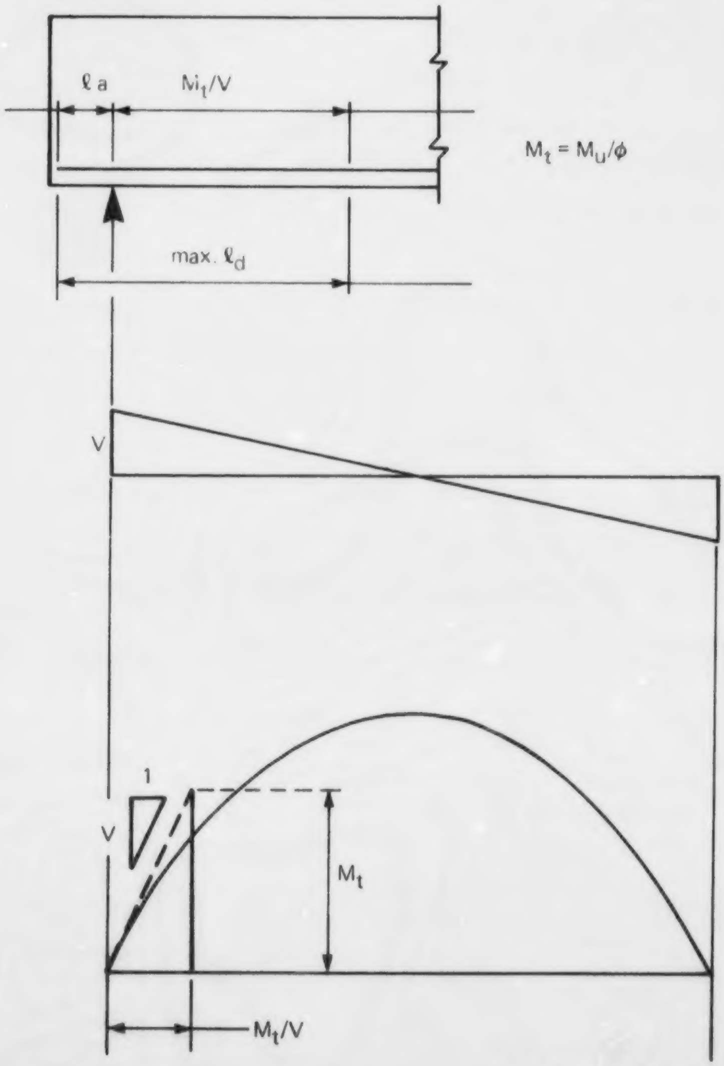
The provisions of this clause are the same as those in Reference 3.

Splicing of Reinforcement **C8 — 10.10**

The provisions of this clause are similar to those in Reference 11.
The factors affecting the lap splice length of reinforcement are:

a) bar size
b) grade of steel
c) concrete strength
d) bar location
e) type of concrete
f) bar spacing and concrete protection
g) lateral reinforcement
h) number of bars spliced at one location
i) minimum splice length

Figure **Concept of Determining Maximum Size Bar at Simple Support** **C8 — 10.1(a)**



Composite Construction **C8 — 11** The provisions of this clause are taken mainly from Reference 4.

T-Girders **C8 — 12** The provisions in this clause are similar to those in Reference 4.

Box Girders **C8 — 13** The provisions in this clause are similar to those in Reference 4.

Concrete Arches **C8 — 14** The provisions in this clause are taken from Reference 4.

Special Detailing Provisions **C8 — 15**

Footings **C8 — 15.1** The provisions in this clause are based mainly on Reference 3. The proportioning for shear should be in accordance with Clause 8 — 8.8.

Piers, Abutments, and Retaining Walls **C8 — 15.2** A typical detail conforming to the provisions of Clause 8 — 15.2.2 is shown in Figure C8 — 15.2.

Mass Concrete **C8 — 15.3** The provisions in this clause are intended for certain bridge components such as gravity walls or abutments and mass pier foundations. The crack control of mass concrete is discussed in Reference 27.

Concrete Piles **C8 — 16** The provisions of this clause are taken mainly from Reference 28.

Concrete Poles **C8 — 17** The provisions for the details of poles are based on the recommendations of References 29, 30, and 31.

The use of coated spiral and longitudinal reinforcement is recommended to improve durability.

Figure **Development of Flexural Reinforcement in a Typical Continuous Beam** **C8 — 10.1(b)**

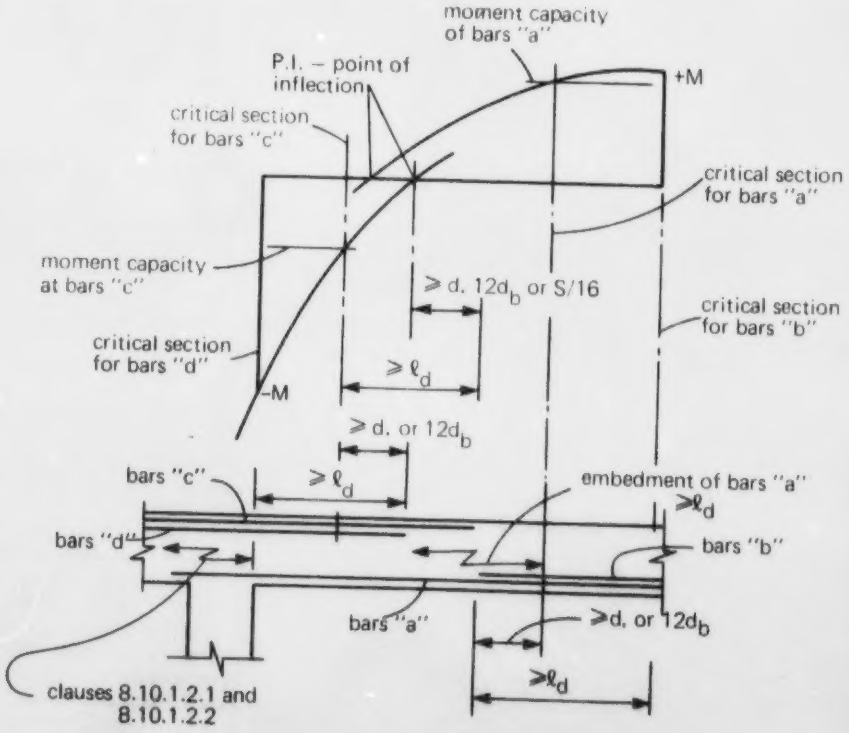
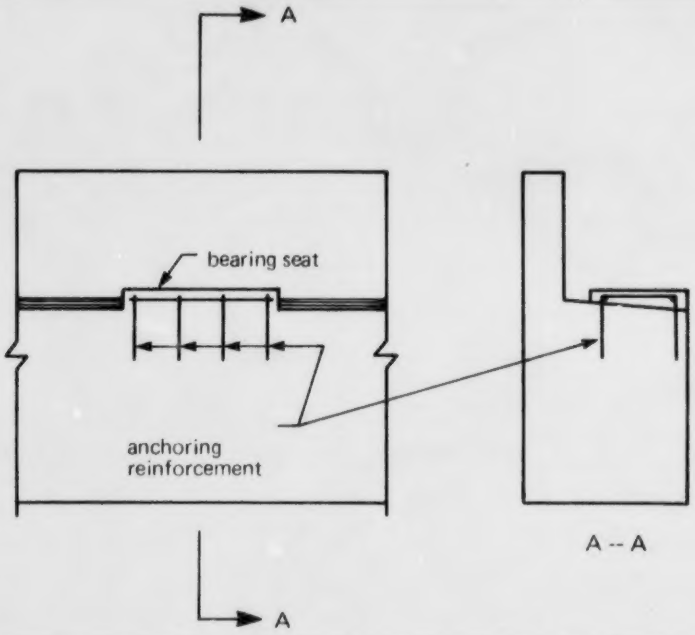

moment capacity of bars "a"
P.I. – point of inflection
critical section for bars "c"
+M
critical section for bars "a"
moment capacity at bars "c"
≥ d, 12d_b or S/16
critical section for bars "b"
critical section for bars "d"
≥ ℓ_d
–M
≥ d, or 12d_b
bars "c"
≥ ℓ_d
embedment of bars "a"
≥ℓ_d
bars "d"
bars "b"
bars "a"
≥d, or 12d_b
≥ℓ_d
clauses 8.10.1.2.1 and 8.10.1.2.2

Figure **Anchorage Reinforcement at Bearing Seat** **C8 — 15.2**



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C9 Prestressed Concrete

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Scope C9—1

General C9—1.1

Section 9 applies to members having an average effective prestress of 1.50 MPa or over.

The provisions of Section 8 also apply to prestressed concrete except where they are specifically excluded, or are in conflict with this section.

Types of Structures C9—1.2

Although prestressing forces can be set up in various ways, the clauses in this section specifically refer to the case where prestress is introduced by high-strength steel tendons.

Use of unbonded tendons is not permitted for new construction because of the difficulty of ensuring adequate corrosion protection, lowering of the ultimate strength relative to that for the equivalent members with bonded tendons, susceptibility to sudden failure and fatigue, and because wide cracks can develop under overloads unless members are adequately reinforced with additional bonded steel. However, unbonded tendons may be used for special applications, e.g., bridge repairs, if approved. In such cases Reference 1 may be referred to as a minimum standard.

Notation C9—3

ACI notation [2] has been used wherever possible in the Code. The following additional notation has been used in the Commentary.

a	a non-dimensional constant used in Clause C9—11.2.
b	a non-dimensional constant used in Clause C9—11.2.
b_c	the width of the compression face of a beam midway between restraint points, mm.
b_s	the width of concrete section in the plane of a potential spalling crack, mm.
CR_t	the amount of prestress loss due to creep that occurs up to t days after transfer, MPa.
c	a non-dimensional constant used in Clause C9—11.2.
F_{sp}	the total spalling force, N.
F_{st}	the total prestressing force at transfer, N.
f_p	the stress at the extreme fibre in tension due to moment acting on the precast portion of a composite section, MPa.
k'	the ratio of creep strain after continuity is established to the elastic strain.
L	the span length, mm.
M_{ct}	the moment computed by applying the loads and/or prestressing force to the completed structure, N·mm.
M_D	the moment due to dead load at transfer, N·mm.
M_o	the modified flexural resistance used in Clause C9—27.7 and given by $(\phi_a/\phi_f)M_u$, N·mm.
M_p	the moment acting on the precast portion of a composite section, N·mm.
M_s	the moment caused by restraint of deformation due to primary prestressing force, N·mm.
M_{sp}	the difference between the moment caused by a linear stress distribution and that caused by the stress distribution at the tendon anchorages, N·mm.
M_{st}	the moment obtained by superimposing moments due to loads and/or prestressing force applied in stages, N·mm.
M_α	the final moment at any section along a structure constructed in stages, N·mm.
n	modular ratio, E_s/E_c.
p	the ratio of the area of prestressing steel to that of the gross concrete area, A_{ps}/A_g.
q	a term used in Clause C9—20.2, MPa.
SH_t	the amount of prestress loss due to shrinkage that occurs up to t days after curing, MPa.
t_o	a deck slab haunch dimension to offset camber of precast prestressed girders, measured at the centreline of bearings, mm.
VS	volume-to-exposed surface area ratio, mm.
X_m	the location of maximum bursting stress measured from the loaded face of the end block, mm.
y''	the second derivative of the equation of the tendon profile, radians per mm.
Z	a parameter given in Clause C9—11.2.1, MPa.
β	the ratio between the depth of the stress block and the depth of the strain neutral axis.
ϵ	the strain in the prestressing steel corresponding to a specific stress.
ϵ_{ce}	the strain induced in the concrete under loads.
ϵ_{cr}	the strain due to creep.
ϵ_{sa}	the increase in strain in the prestressing steel compatible with ϵ_u.

ϵ_{se}	the strain imparted to the steel by prestressing.
ϵ_{sh}	the strain due to shrinkage.
ϵ_{su}	the ultimate tensile strain in the prestressing steel.
ϵ_{u}	the ultimate compressive strain in concrete.
η	a creep factor to account for the volume-to-exposed surface area ratio in Clause C9—9.2.3.2(a).
θ	the rotation of the discontinuous end of a beam due to creep and shrinkage deflections, rad.
σ	the stress in prestressing steel corresponding to a specific strain, MPa.
τ	a creep factor to account for the age of moist-cured concrete at transfer in Clause C9—9.2.3.2(a).
ϕ_a	the performance factor for compression members.
ϕ_f	the performance factor for flexural members.

Limit States C9—4

Serviceability Limit States C9—4.2

This clause specifies the various serviceability limit states to be considered.

C9—4.2.1

Except for partially-prestressed members, fatigue at present is not considered to be a serviceability limit state Type I for prestressed concrete members provided they are not subjected to repetitive loads in excess of the cracking loads. However, a very limited number of tests [3] on prestressed concrete girders, initially loaded to develop cracks at controlled locations, indicate that the fatigue life of girders proportioned for a tensile stress of 0.80 f_r may be lower than that of girders proportioned for zero tension. Recently, a design equation based on strand fatigue test data [75] for allowable fatigue stresses in uncracked girders has also been proposed. There are also indications that the fatigue life of members prestressed with low-relaxation steels can be higher than that of members prestressed with stress-relieved steels [4]. With further investigations, fatigue in future may become a consideration in the design of prestressed concrete members subject to repetitive tensile stresses. The fatigue requirements for partially-prestressed members are given in Clause 9—8.3.

C9—4.2.2

The tensile stresses in the concrete are limited so as to minimize the probability of crack formation and fatigue failure. The compressive stresses in the concrete are limited so as to avoid micro-cracking. Deformations affect the function of a structure when:

a) the clearances are reduced below those specified in Section 1;
b) water ponds on the deck; or
c) the slopes of members at deck joints result in poor riding quality.

General Considerations C9—5

Design C9—5.1

Prestressed members should be investigated at all stages that may be significant. The main stages generally are:

1/ at transfer of prestress;
2/ during handling, transportation, erection and construction;
3/ at serviceability limit states after allowances for all prestress losses;
4/ at ultimate limit state.

For structures constructed in stages, the effects of loads prior to and after redistribution of load effects due to creep and shrinkage should be investigated in the completed structure.

Buckling C9—5.3

This clause refers to local and overall instability of prestressed members. An investigation of the stability of a member, or part thereof, requires the use of second order analysis.

Precautions should be taken to prevent buckling of post-tensioned members in the regions between the contact points where the tendon makes contact with the member intermittently. If the tendon is in complete contact with the member being prestressed, or is an unbonded tendon in a duct only slightly larger than the tendon, it is not possible to buckle the member under the prestressing force being introduced. However, if the duct is larger than the unbonded tendon, lateral deformation will be induced during prestressing. These deformations, the calculation of which has been discussed by Wiiby [5], should be considered.

Thin webs under high shear or compression and flanges in compression are susceptible to buckling between supports. It is recommended that the height-to-thickness ratio of the web of a member should not be greater than 8 [73] in regions where the shear force, V, exceeds 0.15 $b_w d f'_c$.

Members may also become unstable during handling and erection. Consideration should be given to the beam geometry, location of lifting points, method of lifting and construction tolerances. Slender beams tend to tip and deflect laterally when being lifted by slings attached to lifting points. The initial tilting, which is due to imperfections in beam geometry and improper location of lifting points, causes lateral bending moments that may lead to cracking and lateral instability of the beam. Methods of estimating the lateral buckling capacity of a member during lifting have been given by Anderson [6], Swann and Godden [7], and Monnier [8].

Particular attention should be given to erection of beams in which the distance between lateral restraints exceeds 60 b_c or 250 b_c^2/d [9].

Minimum Prestress **C9 — 5.4** The choice of 1.50 MPa is based on experience.

Basic Assumptions

C9 — 6

At Serviceability Limit States **C9 — 6.1** Loss in concrete area due to openings for ducts, coupler sheaths or transition trumpets prior to grouting becomes significant when such loss exceeds 5% of the gross area.

Specified Strengths

C9 — 7

Concrete **C9 — 7.1**

C9 — 7.1.1 Concretes with specified strengths greater than 40 MPa for factory produced members and 35 MPa for cast-in-place members require better than normal quality control and/or aggregates. The specified strength has been limited to 55 MPa since the rectangular stress block does not accurately predict behaviour when concrete strengths exceed this value [10].

The minimum specified strength has been set at 30 MPa as it is generally not economical or desirable to use lower strengths for prestressed concrete work.

In localities where it is difficult to achieve the specified strength of 35 MPa consistently, the design of cast-in-place members should be based on a lower specified strength.

C9 — 7.1.2 For pretensioned concrete, the minimum concrete strength at transfer has been specified to ensure adequate bond between the strands and the concrete. For post-tensioned concrete it is sometimes desirable to provide prestress at an early age to control shrinkage cracks. Particular attention should be given to the size and details of the anchorages.

Grout **C9 — 7.2**

C9 — 7.2.1 The specified strength of grout has been set at 35 MPa as this is the practical limit for field applications.

C9 — 7.2.2 The restriction on the application or removal of loads after grouting and prior to the grout reaching a strengh of 20 MPa has been imposed to avoid possible bond failure between grout and tendons.

Post-Tensioning Anchorages and Couplers **C9 — 7.4** The 95% of specified strength of tendon for anchorages, and 100% for couplers has been specified instead of the generally accepted values of 90% and 95%, respectively, since the provisions of the Code allow jacking stresses of up to 0.85 f_{pu}. The dimensions of anchorages are dependent on the jacking force and the concrete strength at transfer and are established by the manufacturers of post-tensioning systems. The provisions in the Code place no restriction on bearing stresses behind post-tensioning anchorages, as a number of structures with anchorage bearing stresses considerably higher than those recommended by ACI Committee 443 [11], have been successfully built in Canada over the years. The bursting and spalling zones should, however, be adequately reinforced in accordance with Clause 9 — 17.1.2.

Anchorages and couplers, located in zones where externally applied loads cause significant changes of stress, should be investigated for fatigue and should meet the requirements given in Reference 12.

Concrete Stress Limitations

C9 — 8 The permissible tensile stresses have been given in terms of the flexural tensile stress at cracking, f_r, to maintain a uniform margin against cracking for both normal-density and low-density concretes.

At Transfer **C9 — 8.1** The permissible stresses at transfer under the action of prestress and self load, are based on Reference 13 except for segmental members, where it is reasonable to allow a small tensile stress if non-prestressed bonded reinforcement is provided in the tension zone across the joints. The permissible stresses at transfer are also applicable to prestressed concrete members during construction.

The required non-prestressed bonded reinforcement should be distributed evenly near the tensile faces of the member.

Under Loads **C9 — 8.2**

The stresses at serviceability limit states Type II loads should include the effects of redistribution of moments due to creep and shrinkage. Creep effects are referred to specifically in Clause 9 — 21.3.

The maximum permissible compressive stress in precast members produced in certified plants has been increased to 0.45 f'_c from the generally accepted value of 0.40 f'_c. The actual concrete strength is controlled by the concrete strength at transfer, and the 28-day strength is generally higher than the specified strength.

The permissible tensile stresses are based on Reference 13 with slight modifications.

The limit on the average compressive stress due to prestress alone, in post-tensioned decks with round voids, is based on Reference 14. The results of the research indicate that there is a critical average stress level in longitudinal compression in voided members, above which the transverse tension due to stress concentrations over the voids can exceed the flexural tensile stress at cracking, f_r.

Post-tensioned bridge decks with round voids and without transverse prestressing have in the past developed longitudinal cracks over the voids. These decks should therefore be prestressed transversely [14, 15, 16, 17]. An effective transverse prestress of about 4.5 MPa in the concrete above the longitudinal voids has been found to be satisfactory for bridges where the individual spans exceed 1.5 times the deck width and the skews are less than 25°. The minimum concrete thicknesses above and below the voids should be 175 mm and 125 mm, respectively. The length of the solid slab at the ends of the deck should be at least equal to the distance from the outermost anchorage to the edge of the deck, so as to disperse the prestressing forces, but be not less than 0.07 of the span length. Abrupt changes in the cross-section should also be avoided.

Tension is not allowed in segmental construction without bonded non-prestressed reinforcement passing through the joints. This is to avoid the separation of the segments at the joints and the eventual formation of potential weak spots where corrosion of prestressing tendons may take place.

Due to poor performance of unprotected bridge decks in the past, no tension is permitted at the top of such decks. As the quality of materials and construction improves, some tension above the top of prestressing steel may be permitted in the future.

Partial Prestressing **C9 — 8.3**

If the tensile stress limits specified in Clause 9 — 8.2(c) are exceeded then it becomes more probable that the prestressed concrete will crack.

Cracking will cause an increase in the fluctuation of steel stress which results from, say, a truck crossing the bridge. To avoid the possibility of a fatigue failure the range of this fluctuation of steel stress is limited to 125 MPa. This limit, which applies to both prestressed and non-prestressed reinforcement, was chosen after consideration of the limits suggested in References 18, 19, and 20.

While recent research [21] indicates that there is no simple, direct relationship between the width of flexural cracks and durability, it still seems probable that excessively wide cracks will increase the likelihood of corrosion of the reinforcement, especially the reinforcement which is parallel to these cracks. One conventional procedure for improving durability is to detail the member in such a way that the calculated crack widths under serviceability limit states Type II loads remain within acceptable limits. It has been suggested [20] that equations developed for prediction of crack widths in reinforced concrete members, such as the expression given in Clause 8 — 6.2.2.2, can be used for prestressed concrete members where strands are used, provided that the steel stress used in these expressions is taken as the increase in steel stress after the condition of zero concrete strain at the level of the tendon is reached. Because of the concern that prestressing steel may be more sensitive to corrosion, the CEB/FIP Code [22] recommends that allowable crack widths for prestressed concrete members be one half of the allowable crack widths for non-prestressed concrete members. More detailed information on the design of partially-prestressed concrete is contained in References 23, 24, and 25.

Prestressing Requirements **C9 — 9**

Stress Limitations **C9 — 9.1**

In order to utilize fully the advantages of low-relaxation steels in terms of reduced relaxation losses and higher proportional limit, the stress limit at transfer has been increased to 0.75 f_{pu}. However, the stress limit at jacking has been increased to 0.85 f_{pu} only for post-tensioned construction for the following reasons.

a) Due to friction loss in post-tensioned members, the higher jacking stress is limited to a short distance from the stressing end.

b) Experience gained in Australia [26], by using a jacking stress of 0.85 f_{pu} for both low-relaxation and stress-relieved steels, and in France [27], for post-tensioning, seems to be favourable in terms of safety and economy.

c) A higher stress level increases the probability of strand breakages during jacking. In pretensioning, a drop in temperature after jacking and prior to placing concrete results in an increase in steel stress. Consequently, in pretensioning, where the strands are exposed, the risks associated with the higher stress level are greater. On the other hand, for post-tensioned construction, where the tendons are enclosed within the concrete, the risks associated with the tendon breakage are low.

The maximum stress of 0.94 f_{py} at jacking is based on Reference 28, and usually only governs for high-strength bars.

Loss of Prestress C9 — 9.2

C9 — 9.2.1 General

Prestress losses depend on material properties, the environment, and stress levels at the application of various loads and special circumstances, such as restraints and temporary prestressing.

Prestress losses are divided into two groups. "Losses at Transfer" take place up to the time the prestressing force is transferred to the concrete element. The "Losses after Transfer" commence at transfer and continue through the life of the structure.

Figure C9 — 9.2.1 is a schematic diagram of stress levels during the life-time of a structure. A simplified method for members constructed and prestressed in a single stage is given in the Code for computing prestress losses; a more detailed method is outlined in Reference 29.

The lump-sum losses given in Table C9 — 9.2.1 are for normal-density concrete and may be used for preliminary design. They are based on the highest allowable transfer stresses in Clause 9 — 9.1 and Grade 1860 Strand. The lump-sum losses do not include anchorage and friction losses in post-tensioned tendons.

Figure **Schematic Diagram of Stress Levels During the Lifetime of a Structure** **C9 — 9.2.1**

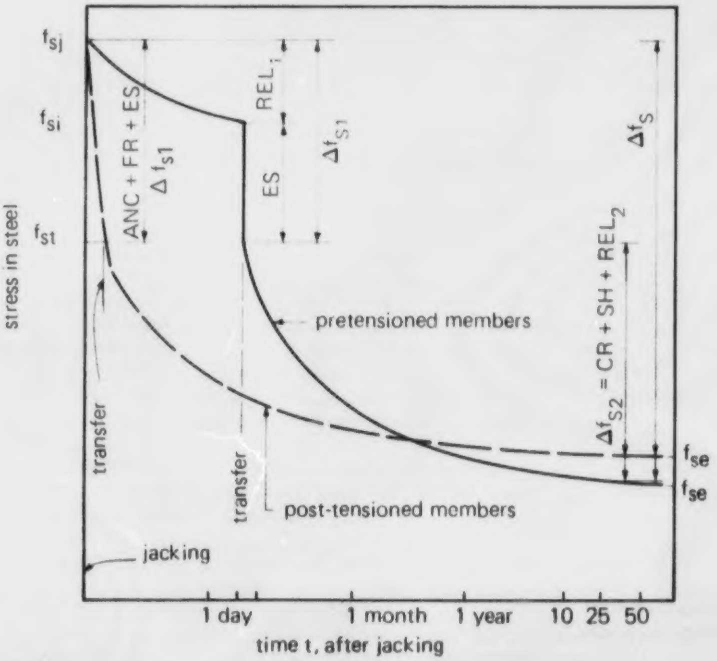


Table **Lump-Sum Losses, MPa** **C9 — 9.2.1**

		Pretensioned		Post-Tensioned	
		SR	LR	SR	LR
At Transfer:	REL_1	75	15	–	–
	ES	125	115	30	30
	Δf_{s1}(MPa)	200	130	30	30
After Transfer:	CR	95	85	70	65
	SH	40	40	35	35
	REL_2	120	25	150	35
	Δf_{s2}(MPa)	255	150	255	135
Total Δf_s:		455	280	285	165

SR = Stress-relieved strand and wire
LR = Low-relaxation strand and wire

In low-density concrete, REL_1 is the same as in normal-density concrete; ES, CR and SH increase roughly in proportion to the ratio of the elastic moduli between the two types of concrete. As a result of the higher plastic strains in low-density concrete, REL_2 decreases slightly. The total prestress losses in low-density concrete having a mass density of 2000 kg/m³ are higher than those for normal-density concrete by the following approximate percentages.

	Type of Steel	
	Stress-Relieved	Low Relaxation
Pretensioned Members	15%	20%
Post-Tensioned Members	10%	15%

Losses for low-density concrete having mass densities lower than 2000 kg/m³ increase roughly at the rate of 35 MPa for every 160 kg/m³ decrease in mass density.

For members constructed and prestressed in multiple stages or for segmental construction, the stress level at the commencement and termination of each stage should be considered.

C9 — 9.2.2 **Losses at Transfer**

C9 — 9.2.2.1 Losses at transfer are due to anchorage slip, friction, pure relaxation of the prestressing steel and the elastic shortening of the concrete at transfer of prestress.

C9 — 9.2.2.2 Where slip at the anchorage is used to control the stress in prestressing steel at transfer, the magnitude of the slip should not be less than the value recommended by the manufacturer of the particular anchorage. A value of slip smaller than that recommended by the manufacturer is difficult to achieve.

C9 — 9.2.2.3 The standard expression for curvature and wobble friction loss is given in the Code and may be generalized to give the stress, $f_{s(x)}$, at any point, x, along the span in terms of the jacking stress. Thus,

$$f_{s(x)} = f_{s_j}[e^{-x(K + \mu y'')}]$$

where y'' is the second derivative of the equation of the tendon profile. The equation of the tendon profile should consider both horizontal and vertical alignment.

For exterior spans of continuous bridges with tendons having a parabolic profile in one plane, a maximum rise, and anchored at the centroid of the section,

$$y'' \simeq \frac{6h}{L^2}$$

For interior spans of continuous bridges with tendons having a parabolic profile with a maximum rise,

$$y'' \simeq \frac{8h}{L^2}$$

For other tendon profiles, the values of y" are given in Reference 30. The wobble and friction factors for strand and wire tendons in rigid steel sheath are based on tests conducted by the Ministry. The values for strand and wire in semi-rigid steel sheaths over 75 mm O.D. and for smooth and threaded bar are taken from Reference 13. The values for strand and wire in semi-rigid steel sheaths up to 75 mm O.D. are based on the manufacturers' recommendations.

C9 — 9.2.2.4 The expressions for pure relaxation of prestressing strand and wire are well established [31]. Pure relaxation loss takes place prior to transfer and pertains only to pretensioned members. The jacking stress, f_{sj}, may be computed from the stress immediately before transfer, f_{si}, from the following expressions:

a) Stress-Relieved Strand and Wire [32]:

$$f_{sj} = \frac{f_{si} - 0.066\, f_{pu} \log(24\, t)}{1 - 0.122 \log(24\, t)}$$

b) Low-Relaxation Strand and Wire:

$$f_{sj} = f_{si}[1 + 0.029 \log(24\, t)] - 0.017\, f_{pu} \log(24\, t)$$

These simplified expressions have been derived from the expressions for relaxation loss in the Code.

C9 — 9.2.2.5 The term f_{cir} in the expression for ES may be expressed as follows:

$$f_{cir} = \frac{F_{st}}{A_g} + \frac{F_{st}e^2}{I} - \frac{M_D e}{I} + \frac{M_s e}{I}$$

where: $F_{st} = f_{st}A_{ps}$

Since f_{st} is limited to 0.70 f_{pu} for stress-relieved steels and 0.75 f_{pu} for low-relaxation steels, the losses may be computed by starting at f_{st} and working upwards to the jacking stress, f_{sj}, and downwards to the effective stress, f_{se}. The stress in prestressing steel immediately before transfer, f_{si}, may be computed directly from the following expression:

$$f_{si} = f_{st}\left[(1 + \gamma) - \frac{M_D}{F_{st}e}(\gamma - np)\right]$$

where: $n = E_s/E_{ci}$

$\gamma = np\,[1 + (e/r)^2]$

The elastic shortening loss may be computed from:

$$ES = f_{si} - f_{st}$$

In post-tensioned members, elastic shortening is different in each tendon and varies according to its sequence in the tensioning process. Hence, the factor (N – 1)/2N is incorporated in the expression for ES [32]. In low-density concrete members, loss in prestress due to elastic shortening is considerably higher than that in corresponding normal-density concrete members, since ES is directly proportional to E_{ci}.

C9 — 9.2.3 **Losses After Transfer**

C9 — 9.2.3.1 Losses after transfer are due to creep and shrinkage of the concrete member and relaxation of the prestressing steel.

C9 — 9.2.3.2 **a) Normal-Density Concrete**

The basic expression for creep given in Reference 33 is modified to include the effect of the mean annual relative humidity, H. The expression within the first set of brackets, namely, $[1.37 - 0.77(0.01H)^2]$, yields approximately unity for H = 70%. This expression is derived from the data given in Reference 29. Creep is also affected by the shape factor, η, of the member and, for moist-cured members, by a function of its age at transfer, τ, computed as follows:

$\eta = 1.14 - 0.0036\, VS \geq 0.68$, and

$\tau = 1.20 - 0.05\, (t)^{2/3}$

The effects of these factors are included in the expression given in the Code by using nominal values of $\eta = 0.8$ and $\tau = 1.0$ at 7 days. The amount of prestress loss due to creep that occurs up to t days after transfer, CR_t, as a function of total creep loss, CR, may be computed from:

$$CR_t = (1 - e^{-0.09\sqrt{t}})CR$$

b) Low-Density Concrete

Due to wide variations in the types and properties of low-density concretes, and the lack of published data regarding creep, it is not possible to formulate expressions for creep loss and, hence, the reference to actual creep data is made in the Code. Creep strain is a function of the modulus of elasticity of concrete. Therefore, the loss of prestress due to creep in low-density concrete members can be 15% to 40% more than in normal-density concrete of equivalent strength. The lower limit pertains to concretes with a mass density of 1900 to 2000 kg/m³ and the upper limit to those with a mass density of 1400 to 1600 kg/m³.

C9 — 9.2.3.3

a) Normal-Density Concrete

The expression for prestress loss due to shrinkage, based on Reference 34, is used and it includes the effects of humidity. At jacking in post-tensioned members, approximately 20% of the strain due to shrinkage has taken place, hence the shrinkage loss for such members is reduced.

The amount of prestress loss due to shrinkage occurring up to t-days after placing of concrete, SH_t, may be computed from:

$$SH_t = (1 - e^{-0.10\sqrt{t}})SH$$

b) Low-Density Concrete

Reference to actual shrinkage data is required in the Code. Shrinkage strain is primarily a function of the porosity of concrete and the type of aggregate. Low-density concrete is more porous than normal-density concrete, and the prestress loss due to shrinkage in low-density concrete is about 10% to 20% higher than in normal-density concrete.

C9 — 9.2.3.4

After transfer, the stress in the prestressing steel changes with time as a result of the effects of creep and shrinkage strains in the concrete and relaxation in the steel. The expressions in Clause 9 — 9.2.3.4 are modified to include the effect of plastic strains in the concrete [32, 35].

The expressions in Clause 9 — 9.2.3.4 can be rearranged into a format found in other codes [13, 34] as follows:

$$REL_2 = C[A - B(CR + SH)]$$

where A, B, and C are variables depending on the type, specified strength, and transfer stress of the prestressing steel.

Table C9 — 9.2.3.4 gives values of A, B, and C for various combinations of the independent variables.

Relaxation losses in high-strength bars are generally lower than those for strand or wire. At present, there are insufficient data to formulate general expressions for computing the relaxation losses in high-strength bars. The relaxation losses should, therefore, normally be based on test data supplied by the manufacturers, provided that approval is obtained for this procedure.

Table **Values of A, B and C for Computation of REL_2** **C9 — 9.2.3.4**

	Type of Strand and Wire			
	Stress-Relieved		Low Relaxation	
Grade	A	B	A	B
1860	140	0.144	42	0.053
1760	133	0.144	40	0.053
1720	131	0.144	39	0.053
f_{st}/f_{pu}	C		C	
0.75			1.00	
0.74			0.95	
0.73			0.90	
0.72			0.85	
0.71			0.80	
0.70	1.00		0.75	
0.69	0.94		0.70	
0.68	0.89		0.65	
0.67	0.83		0.60	
0.66	0.78		0.55	
0.65	0.72		0.50	
0.64	0.67			
0.63	0.61			
0.62	0.56			
0.61	0.50			
0.60	0.44			

Performance Factors **C9 — 10**

The performance factors are computed from procedures given in Reference 36.
For compression members with flexure, it is reasonable to increase the value of the performance factors as shown in Figure C9 — 10.

Proportioning of Sections for Flexure **C9 — 11**

General **C9 — 11.1**

C9 — 11.1.1

The provisions in this clause are taken from the literature and other codes [13, 34, 37]. The expressions for over-reinforced sections given in Clauses 9 — 11.1.1(b) and 9 — 11.1.2(b) are conservative. Over-reinforced sections are not preferred. An over-reinforced section may be modified to become an under-reinforced section by change in geometry or by addition of non-prestressed compression reinforcement to ensure ductility of the section.

C9 — 11.1.2

The neutral axis falls outside the flange when:

$$h_f < d\rho_p f_{ps}/0.85\,\beta f'_c$$

where: $\beta = 0.85 - 0.008\,(f'_c - 30)$

Steel Stress **C9 — 11.2**

C9 — 11.2.1

Bonded Tendons

A method based on strain compatibility at failure [37, 38] using representative stress-strain curves is recommended for computing the value of f_{ps}. In a bonded member, the total strain in the prestressing steel at failure, ϵ_{su}, is given by:

$$\epsilon_{su} = \epsilon_{se} + \epsilon_{ce} + \epsilon_{sa}$$

where: $$\epsilon_{se} + \epsilon_{ce} = \frac{f_{se}}{E_s}(1 + \gamma)$$

and: $$\epsilon_{sa} = \left[\frac{0.85\,f'_c\beta}{\rho_p f_{ps}} - 1\right]\epsilon_u$$

Assuming $\epsilon_u = 0.003$, for rectangular sections.

Figure **Variation of Values for Flexural Members With Small Axial Compression** **C9 — 10**

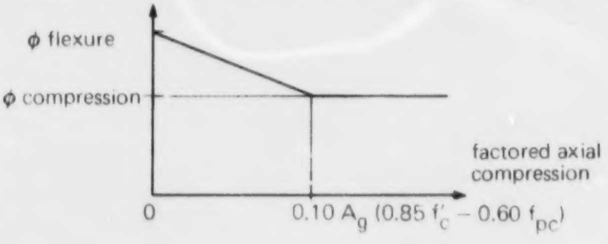
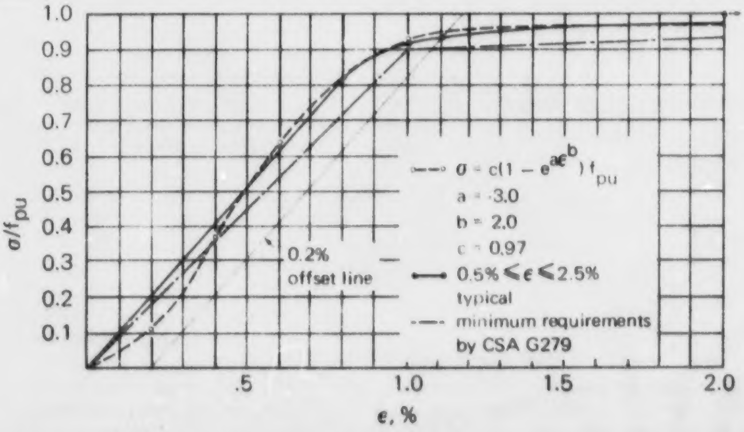


Figure **Low-Relaxation Strand Size Designation: 13** **C9 — 11.2.1(a)**



$$\epsilon_{su} = \frac{f_{se}}{E_s}(1 + \gamma) + 0.003\left[\frac{0.85\, f'_c \beta}{\rho_p f_{ps}} - 1\right]$$

where β is given in Clause C9 — 11.1.2. In the absence of representative stress-strain curves for prestressing steel, the following expression may be used for strands and wires:

$$\sigma = c(1 - e^{a\epsilon^b})\, f_{pu}$$

where σ is stress and ϵ is the corresponding strain in percent. The parameters a, b, and c pertaining to Grades 1720, 1760, and 1860 Strand, respectively, are given in Table C9 — 11.2.1. These parameters are based on stress-strain curves shown on Figures C9 — 11.2.1(a) to C9 — 11.2.1(d). Where the stress-strain curves are likely to be different from those shown, the parameters should be adjusted. The following compatibility expression for rectangular sections is derived by solving the equations for ϵ_{su} and σ.

Table **Values of Parameters a, b, and c for Strand** **C9 — 11.2.1**

		Strand Size Designation	
Type of Steel	Factors	13	15 and 16
Stress-Relieved	a	-3.0	-2.5
	b	2.0	1.5
	c	0.95	0.98
Low-Relaxation	a	-3.0	-2.6
	b	2.0	1.5
	c	0.97	0.97

Figure **Low-Relaxation Strand Size Designation: 15 and 16** **C9 — 11.2.1(b)**

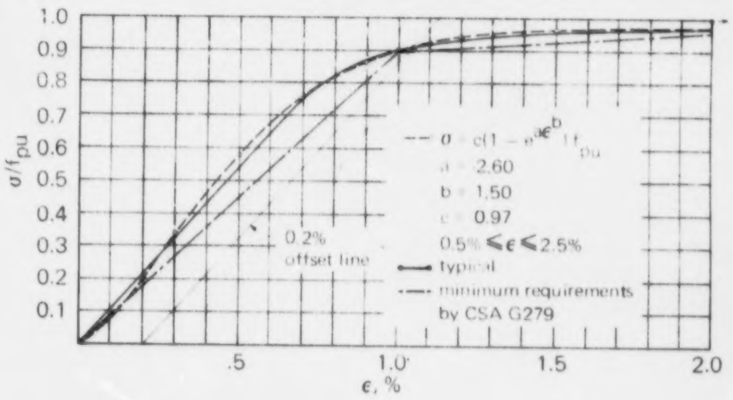
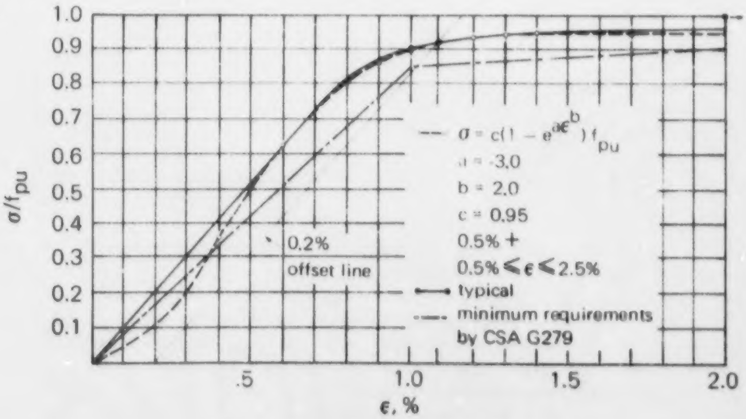


Figure **Stress-Relieved Strand Size Designation: 13** **C9 — 11.2.1(c)**



$$-\ln\left[1-\frac{f_{ps}}{cf_{pu}}\right]=a\left[100\frac{f_{se}}{E_s}(1+\gamma)+0.3\left(\frac{0.85\,f'_c\beta}{\rho_p f_{ps}}-1\right)\right]^b$$

where the quantity within the square bracket is the strain, ϵ.

The value of f_{ps} may be found by iteration, using an initial value computed from Clause 9 — 11.2.1.

Similarly, for flanged sections, the compatibility expression is:

$$-\ln\left[1-\frac{f_{ps}}{cf_{pu}}\right]=a\left[100\frac{f_{se}}{E_s}(1+\gamma)+100\epsilon_{sa}\right]^b$$

where: $\epsilon_{sa} = \left[\epsilon_u\dfrac{\beta}{\dfrac{t}{d}\left(1-\dfrac{b}{b_w}\right)+\dfrac{b}{b_w}\left(\dfrac{f_{ps}\rho_p}{0.85\,f'_c}\right)}-1\right]$

When $b = b_w$, the above expression for ϵ_{sa} reduces to that for a rectangular section.

In high tensile strength alloy bars, where the yield point is well defined, the stress-strain relationship may be assumed to be elastic-plastic such that $\sigma = E_s\,\epsilon \leq f_{py}$.

The following expression for rectangular sections yields f_{ps} directly:

$$f_{ps}=Z\left[1+\sqrt{1+\frac{0.85\,f'_c\,E_s\beta\epsilon_u}{\rho_p Z^2}}\right]\leq f_{py}$$

where: $Z = 0.5[f_{se}(1+\gamma)-E_s\epsilon_u]$

Similarly, for flanged sections reinforced with high-strength bars, the compatibility expression is:

$$f_{ps}=E_s\epsilon_{sa}+f_{se}(1+\gamma)$$

where the value of ϵ_{sa} is the same as given for flanged sections prestressed with strand or wire. The expression in Clause 9 — 11.2.1(a) for steel stress at ultimate limit state for stress-relieved steel is derived in Reference 38. The corresponding expression for low-relaxation steel was derived in a similar manner.

C9 — 11.2.2 **Unbonded Tendons During Construction**

Members with unbonded tendons are not permitted in completed structures for the reasons given in Clause C9 — 1.2. During construction, however, stages will exist where the tendons are unbonded.

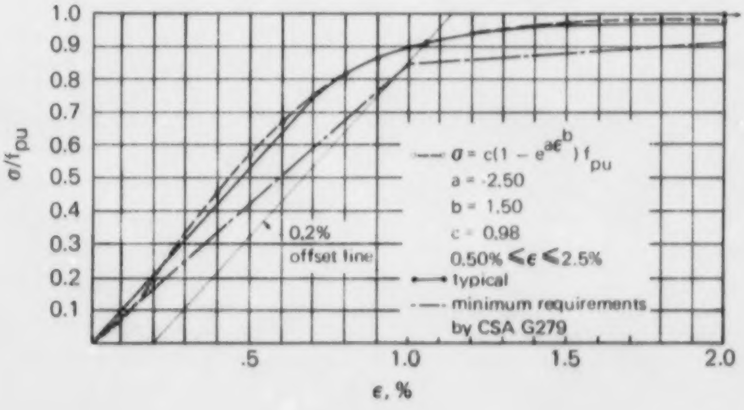
When members with unbonded tendons are loaded, slip occurs between concrete and the prestressing steel. Hence, strain compatibility between concrete and steel cannot be assumed. The expression for f_{ps} is taken from Reference 39.

Non-Prestressed Reinforcement — C9 — 12

Contribution to Factored Resistance — C9 — 12.2

When non-prestressed reinforcement is used to increase the factored resistance of a section, the reinforcement should be detailed in accordance with Section 8.

Figure **Stress-Relieved Strand Size Designation: 15 and 16** **C9 — 11.2.1(d)**



Shrinkage and Temperature

C9 — 12.3

A minimum ratio of non-prestressed reinforcement has been specified for post-tensioned members to control cracking due to shrinkage and temperature gradients before prestressing [40].

Maximum and Minimum Reinforcement

C9 — 13

Maximum Reinforcement

C9 — 13.1

The 0.30 limit is recommended to ensure ductile behaviour at the ultimate limit state.

Minimum Reinforcement

C9 — 13.2

C9 — 13.2.1

It is desirable that members contain a sufficient amount of reinforcement to ensure that a reserve of strength exists after initial cracking. If the members do not contain enough reinforcement they may fail abruptly with rupturing of the steel immediately after cracking. The factor 1.25 is chosen to make allowance for likely variations in the prestressing force and in the strength of the materials.

The cracking moment should be computed considering the effective prestress. It may be computed from:

a) monolithic sections

$$M_{cr} = \frac{I}{y_t}(f_{pe} + f_r)$$

b) composite sections

$$M_{cr} = M_p + \frac{I}{y_t}(f_{pe} + f_r - f_p)$$

C9 — 13.2.2

This clause extends to prestressed concrete the traditional philosophy used in reinforced concrete design — that is, if the factored resistance is increased by one third the probability of a sudden failure will be reduced.

Proportioning of Sections for Compression

C9 — 14

The provisions for the design of prestressed concrete compression members are based on the magnified moment approach and are taken from Reference 41. Further refinements of this approach are given by Nathan [74].

Proportioning of Sections for Shear and Torsion

C9 — 15

The provisions for the shear resistance of sections are similar to those of ACI 318 [28] and CAN A23.3 [39], while the provisions for torsional resistance are those contained in CAN A23.3 [39]. However, the design equations have been written in terms of sectional resistance. These provisions are discussed in the relevant commentaries [42, 43], and hence in this Commentary reference is made only to clauses which differ from those of the above codes.

General Reinforcement Requirements

C9 — 15.1

In flexural members subjected to repetitive loading, there is a possibility of diagonal tension cracks forming at appreciably smaller stresses than under static loading. It is thus desirable to provide minimum shear reinforcement in all bridge members with the exception of solid slabs. Solid slabs are exempted from the minimum shear reinforcement requirements because it is assumed that their ability to redistribute load in the transverse direction will provide sufficient ductility.

An effective stress in the prestressing steel of at least 40% of its ultimate tensile strength is used to classify a member as a prestressed member for shear and torsion. The 40% level appears to be based on data obtained from tests [44].

Shear Resistance Provided by Concrete

C9 — 15.3

The value of V_c in Clause 9 — 15.3.1 is the lower limit on V_c as specified in the corresponding clause in References 28 and 39.

The transfer length referred to in Clause 9 — 15.3.3 is discussed in the Commentary to Clause 9 — 16.

Effect of Post-Tensioning Ducts in Webs

C9 — 15.5

The use of an equivalent web width reflects the reduction in the web crushing capacity of a section due to the presence of post-tensioning ducts in the web. The equivalent web widths are those proposed by Leonhardt [45]. Work at Queen's University [46] has confirmed the validity of using these equivalent web widths in conjunction with the conventional 45° truss model. However, this work has also indicated that use of these specified equivalent web widths may predict low values of the web-crushing capacity when used in more sophisticated truss models where the angle of inclination of diagonal compression in the web is taken less than 45°.

For sections having tapered webs, the minimum web width should be used in computing shear resistance.

In beams with webs containing post-tensioning ducts, consideration should be given to the possibility of early diagonal cracking in the vicinity of the duct and of splitting in the web above and below the duct. Provision of spiral reinforcement around the ducts is known to enhance the web-crushing capacity of the web [46]. However, at present, there are insufficient research data to formulate specific recommendations.

Slabs and Footings **C9 — 15.6**

Test data [47] show that the shear resistance of a prestressed concrete two-way slab with bonded reinforcement is conservatively estimated by the equation in this clause.

Members Subjected to Torsion **C9 — 15.7**

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressed reinforcement may be taken into account by treating it as equivalent to an area of reinforcing steel with a yield force equal to the effective prestressing force.

Alternative Methods of Proportioning for Bending, Shear, and Torsion **C9 — 15.8**

This clause enables the shear and torsional resistance of prestressed members to be determined by rational theoretical models rather than by the empirical procedures of Clauses 9 — 15.2 to 9 — 15.7. A design procedure using such a model, namely a variable-angle space truss, has been proposed by Collins and Mitchell [48].

Development Length of Pretensioning Strand

C9 — 16

The requirements are intended to provide bond integrity for the load capacity of the member. The expression for the development length is valid for concrete transfer strengths between 20 MPa and 50 MPa and is taken from a report by Zia and Mostafa [49].

The initial function of bond is to transfer the prestress force from the pretensioned reinforcement to the concrete member. The bonded length of strand needed for such transfer is known as the transfer length. When a member is subjected to loads in the range from serviceability limit state Type II to ultimate limit state, the stress in the prestressed reinforcement increases substantially beyond the prestress level and bonded embedment length beyond the transfer length is required to develop these increased steel stresses. This length is referred to as the flexural bond length. The sum of the transfer length and the flexural bond length is known as the development length.

The expression for development length can be written as:

$$L_d = L_t + L_b$$

where, L_t, the transfer length, is given by:

$$L_t = 1.5 \frac{f_{si}}{f'_{ci}} d_b - 117$$

and, L_b, the flexural bond length, is given by:

$$L_b = 0.18 (f_{ps} - f_{se}) d_b$$

For design purposes, the stress in the strand may be assumed to vary linearly from zero at the position where bonding starts to f_{se} at a distance L_t from that point, and then linearly to f_{ps} over the adjacent distance L_b. The development length given in this clause is longer than that given in ACI 318 [28] but less than that proposed by Martin and Scott [50].

When strands are debonded for some distance into the end of a member, both stress transfer and flexural bond embedment may take place in a concrete region subjected to tension, before ultimate limit state is reached. To ensure adequate bond, it is recommended [51] that in such cases the development length specified in this clause be doubled. When tension does not occur in the precompressed tensile zone under loads at serviceability limit states Type II, tests [3] indicate that the use of one development length is adequate.

Anchorage Zones, Couplers and Ducts

C9 — 17

Anchorage Zone Reinforcement **C9 — 17.1**

C9 — 17.1.1

Bursting and spalling cracks can occur due to transverse tensile forces in the anchorage zone.

Bursting cracks tend to form along the tendon axis of post-tensioned members, and start at a distance of about 0.5 X_m from the loaded face and extend to a distance equal to the depth of the end block, a_2. Bursting cracks are not likely to occur in pretensioned beams.

Spalling cracks start at the end face of the member, some distance removed from the tendon axis, and propagate approximately parallel to the longitudinal axis of the member. They are likely to occur both in pretensioned and post-tensioned members. Reinforcement should be provided in the end zones to prevent the occurrence of bursting and spalling cracks. The amount of reinforcement should be based on the force at jacking in post-tensioned members, and the force after transfer in pretensioned members.

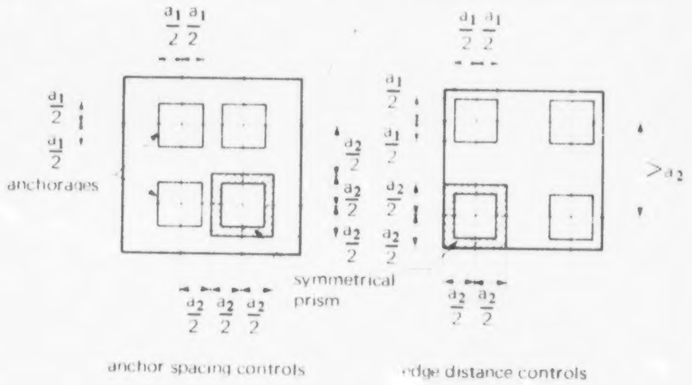
C9 — 17.1.2

End blocks should be provided to distribute concentrated prestressing forces at the ends of members with thin webs. The width of the end block for I-beams should be approximately equal to the width of the narrower flange and the length of the end block should be at least equal to 0.75 h or 600 mm, whichever is greater.

The expressions given in the Code are based on curves given in Reference 52, which are the results of three-dimensional stress analyses. In both centrally- and eccentrically-loaded end blocks, the maximum bursting force and the associated maximum transverse tensile stresses occur along the axis of the tendon. Consequently, for a single anchorage, the symmetrical prism concept may be utilized for assessing bursting forces and stresses as shown in Figure C9 — 17.1.2(a).

The maximum transverse tensile bursting stress, f_{bs}, is largest at the axis of the prism and gradually decreases towards the edges. However, for values of $a_1/a_2 \geq 0.3$, this variation may be neglected and a constant stress may be assumed over the width of the prism. The stress variation becomes more pronounced for values of $a_1/a_2 < 0.30$. The effect of the transverse bursting stress beyond a distance a_2 from the face of the loaded area becomes negligible. The total bursting force is computed from the area within the curve shown in Figure C9 — 17.1.2(b), and acts in the two principal directions. The bursting force, F_{bs}, given in the Code is slightly higher than that calculated from this area. However, the value of F_{bs} is similar to those given in References 53 and 54.

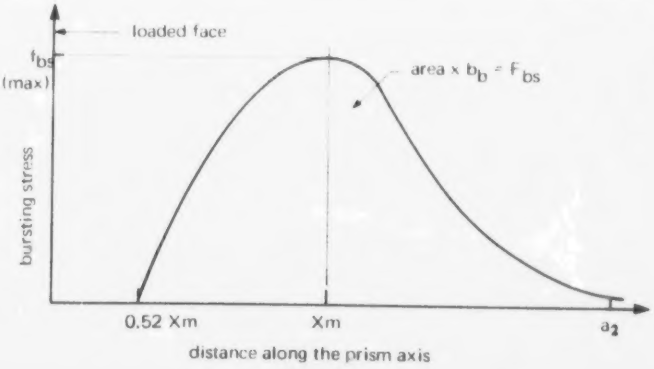
Figure **Symmetrical Prism Concept** **C9 — 17.1.2(a)**



anchor spacing controls

edge distance controls

Figure **Bursting Stress Distribution** **C9 — 17.1.2(b)**



Spalling cracks start at the end face of the member some distance removed from the tendon axis, and propagate parallel to the longitudinal axis of the member. The spalling force and the reinforcement required may be determined using the methods described in References 8, 54, 55 and 56. As a close approximation, the critical location of spalling cracks may be determined by finding the plane on which the difference in moment, M_{sp}, between that caused by linear stress distribution in the section and that caused by the stress at the tendon anchorages is a maximum.

The spalling force, F_{sp}, for rectangular sections can be computed from:

$$F_{sp} = \frac{2M_{sp}}{h}$$

as shown in Figure C9 — 17.1.2(c).

The maximum spalling stress, f_{sp}, computed from:

$$f_{sp} = \frac{6M_{sp}}{b_s h^2}$$

should not exceed 0.50 f_r.

Special consideration should be given where it is not possible to provide end blocks behind anchorages, such as in the webs of segmental box girders [57].

C9 — 17.1.3 The requirements in this clause are based on Reference 34 except that the distance over which the vertical stirrups are distributed has been reduced to 0.20 h as given in Reference 11.

Detailing Requirements of Post-Tensioning Anchorages and Couplers **C9 — 17.2** Anchorages and end fittings should be located in recesses. After grouting, the recesses should be cleaned and filled with concrete of the same quality as the member. In recesses larger than 300 mm square, wire mesh should be placed before concreting.

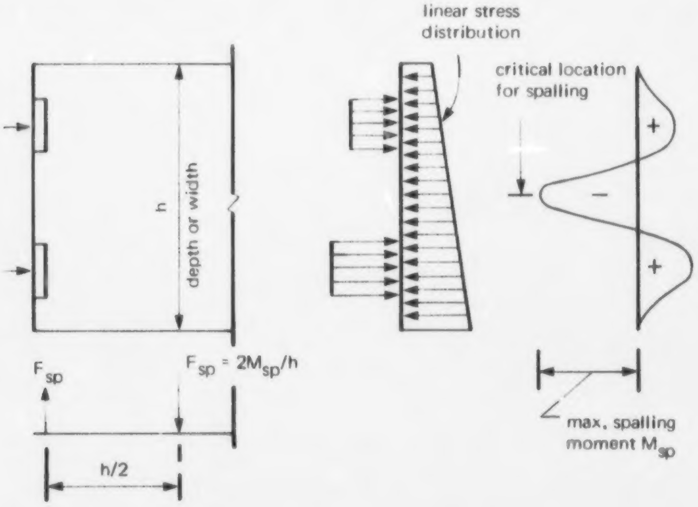
Post-Tensioning Ducts **C9 — 17.3**

C9 — 17.3.1 Ducts should be provided with vents at all high points and at the anchorages, so as to facilitate grouting. Vents should also be provided at low points for construction in cold weather, to avoid the possibility of trapped moisture freezing in the ducts. The location of the vents should be shown on the plans.

C9 — 17.3.2 Post-tensioning ducts are usually grouted under a pressure of 700 kPa. The ducts should be watertight when tested under an internal water pressure of 350 kPa.

C9 — 17.3.3 This clause is based on Reference 34.

Figure **Spalling in Post-Tensioned Members** **C9 — 17.1.2(c)**


linear stress distribution
critical location for spalling
+
−
+
h
depth or width
F_{sp}
$F_{sp} = 2M_{sp}/h$
h/2
max. spalling moment M_{sp}

C9 — 17.3.4 and **C9 — 17.3.5**

The wall thicknesses and bend radii are based on current practice in Ontario.

For segmental construction, it is usually necessary to bend rigid ducts to radii as small as 3 m. In such cases, short lengths of ducts are pre-bent to standard radii using special procedures.

Rigid ducts less than 40 mm in diameter are not readily available and the manufacturers should be consulted prior to specifying them.

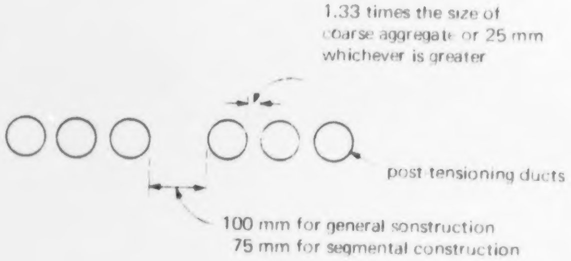
Concrete Protection for Reinforcement **C9 — 18**

In corrosive or marine environments, or in unusually severe exposure conditions, the minimum concrete covers should be increased or other protection provided. In order to achieve the required minimum covers, the average concrete covers and the tolerances for placing the prestressing steel which can be achieved under normal conditions are as shown in Table C9 — 18.

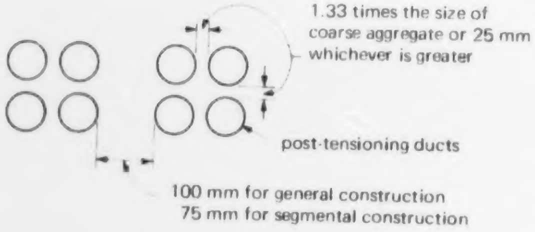
Table **Average Concrete Covers and Tolerances for Placing of Prestressing Steel** **C9 — 18**

		Average Cover mm	Tolerance mm
a)	Pretensioning Strands		
i)	tops of decks	80	± 5
ii)	elsewhere	45	± 5
b)	Post-Tensioning Ducts		
i)	tops of decks		
	longitudinal ducts	130	±15
	transverse ducts		
	$d_d \leq 55$ mm	85	±10
	$d_d > 55$ mm	130	±15
ii)	elsewhere	55 or ($0.50\, d_d + 15$), whichever is greater	±15
iii)	members cast against earth	90	±15

Figure **Recommended Spacing of Ducts** **C9 — 19.2**


1.33 times the size of coarse aggregate or 25 mm whichever is greater
post-tensioning ducts
100 mm for general construction
75 mm for segmental construction

(a) Ducts in a Single Horizontal Plane


1.33 times the size of coarse aggregate or 25 mm whichever is greater
post-tensioning ducts
100 mm for general construction
75 mm for segmental construction

(b) Ducts in Two or More Horizontal Planes

Spacing of Strands and Ducts **C9 — 19**

Pretensioning Strands **C9 — 19.1**

Strands should be located on a 50 mm grid at the ends of members to suit the bulkheads and abutments used by the precast industry. In slabs and box girders with hand-holes for transverse post-tensioning, the strands should be located to provide a minimum cover of 13 mm at the hand-hole.

Post-Tensioning Ducts **C9 — 19.2**

Adequate space should be provided between groups of ducts, as shown in Figure C9 — 19.2, to facilitate placing and vibrating the concrete. For precast or cast-in-place segmental construction, the minimum horizontal space between groups of ducts has been reduced because the segments are usually constructed to close tolerances.

Bundling **C9 — 19.3**

This clause has been adopted from Reference 13.

Directional Change in Prestressing Steel **C9 — 19.4**

Changes in direction of prestressing steel, either in plan or in elevation, will induce thrusts and stresses in the surrounding concrete. The concrete should be capable of resisting the resultant tensile and compressive stresses and non-prestressed reinforcement may be required to counteract the thrust. The non-prestressed reinforcement should be placed in such a way as to prevent bursting or splitting. Thin curved webs should be reinforced to prevent vertical splitting.

An example of thrust due to directional change of prestressing steel occurs when tendons in the web are flared to accommodate anchorages in the end block as shown in Figure C9 — 19.4(a). The thrust can be computed as shown in Figure C9 — 19.4(b).

Figure **Thrust Due to Flaring of the Prestressing Steel** **C9 — 19.4(a)**

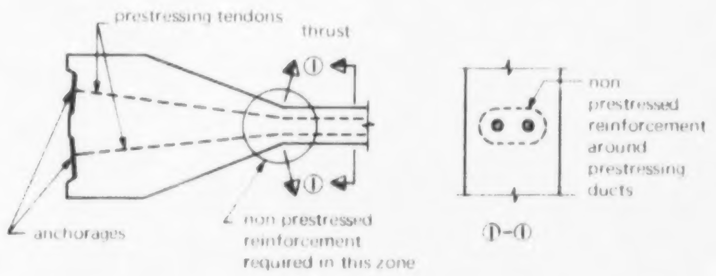
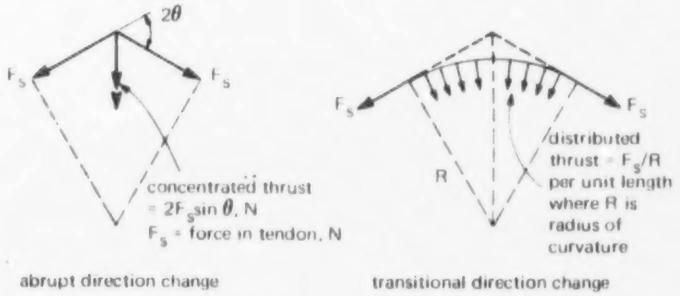

prestressing tendons
thrust
①
①
anchorages
non prestressed reinforcement required in this zone
non prestressed reinforcement around prestressing ducts
①-①

Figure **Magnitude of Thrust** **C9 — 19.4(b)**


2θ
F$_s$
F$_s$
concentrated thrust = 2F$_s$sin θ, N
F$_s$ = force in tendon, N
abrupt direction change
F$_s$
F$_s$
R
distributed thrust = F$_s$/R per unit length where R is radius of curvature
transitional direction change

Deformations **C9 — 20**

General **C9 — 20.1**

Rotation at bearings should be investigated to ensure adequate clearance between the underside of superstructure and the edge of the bearing seat as shown in Figure C9 — 20.1.

The modulus of elasticity of concrete at time of interest should be used to compute instantaneous deformations.

Deflections **C9 — 20.2**

The expression in Clause 9 — 20.2 for time-dependent deflection is derived from a multi-interval rate of creep method [58] similar to the one used to compute prestress losses. The expression is a modified form of:

$$\delta_u = \delta_i \left[1 + \frac{[0.145q\epsilon_{cr} + \epsilon_{sh}][1-0.0725\epsilon_{cr}E_c\gamma(1-\gamma) - \epsilon_{sn}]}{q/E_c} \right] \left[1 - \frac{\Delta f_{s2}}{f_{si}} \right]$$

where:

$\epsilon_{cr} = CR/E_s$

$\epsilon_{sh} = SH/E_s$

$$q = f_{si} \left[\frac{\gamma}{n} - \left(\frac{e_D}{e} \right) \left(\frac{\gamma}{n} - p \right) \right] = ES/n$$

and the constants 0.145 and 0.0725 have the dimension of 1/MPa. The following expression may be used to obtain preliminary estimates of δ_u:

$$\delta_u = \delta_i \left[1 + 0.90 \left(0.145 \frac{CR}{n} - \frac{SH}{E_s} \right) \right] \left[1 - \frac{\Delta f_{s2}}{f_{si}} \right]$$

where the constant 0.90 is dimensionless.

The following load stages as shown in Figure C9 — 20.2(a) should be considered in deflection calculations:

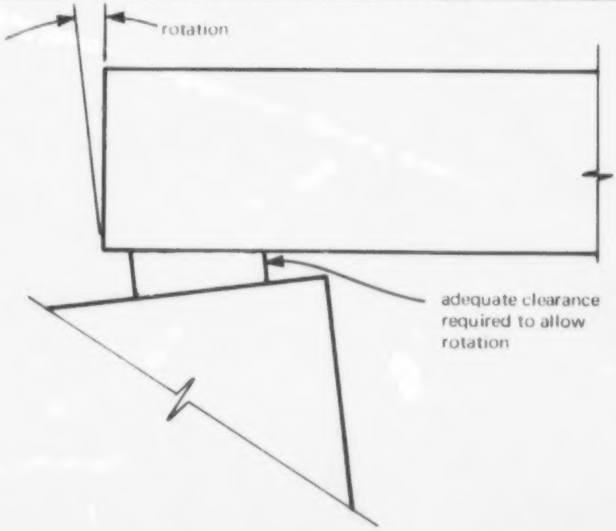
a) prestress plus girder dead load at transfer and prior to casting of deck slab;
b) prestress plus girder and slab dead load; and
c) prestress when all losses have occurred plus total dead load.

To allow for variation in camber of girders at the time of casting the deck slab, the following haunch dimensions may be used, as shown in Figure C9 — 20.2(b), in computing the screed elevations.

Girder Type	t_o
CPCI 900	40 mm
CPCI 1200	50 mm
CPCI 1400	75 mm
Box Girders	50 mm to 75 mm

Rotations may be computed by assuming that the profile of the deflected structure is a parabola.

Figure **Clearance Requirement for Rotation** **C9 — 20.1**



Statically-Indeterminate Structures C9 — 21

General **C9 — 21.1**

Some redistribution of moments can occur in statically-indeterminate prestressed concrete members as their ultimate capacity is approached, but this redistribution can generally be neglected for bridge members.

Secondary Effects **C9 — 21.2**

The secondary effects induced by prestressing can be significant when compared with primary efects.

Effect of Creep **C9 — 21.3**

In statically-indeterminate structures constructed in stages from precast or cast-in-place elements which are made continuous after being subjected to permanent forces as separate elements, the distribution of force effects within the completed structure changes during the subsequent life of the structure [59, 60, 61]. The distribution approaches that which would have occurred if the forces applied to the individual elements in stages before establishing continuity had been applied to the structure after establishing continuity. This applies regardless of whether the permanent forces are prestress forces or forces due to dead load. The final moment at any section along such a structure may be computed from the following relationship:

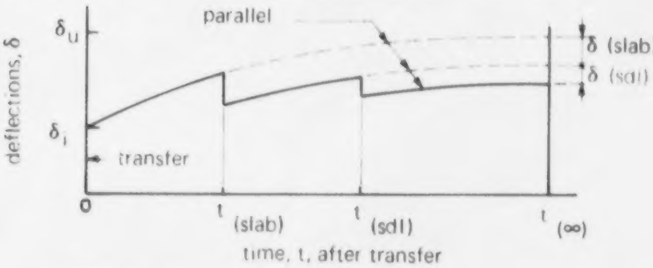
$$M_{\infty} = M_{st} + (M_{ct} - M_{st})(1 - e^{-k'})$$

The creep factor, k', is affected by relative ambient humidity, volume-to-exposed-surface ratio, age at loading, method of curing, type of cement and water-cement ratio, and may be computed from data in Reference 61.

There is no redistribution of load effects due to creep in statically-indeterminate structures erected or constructed in their final configuration.

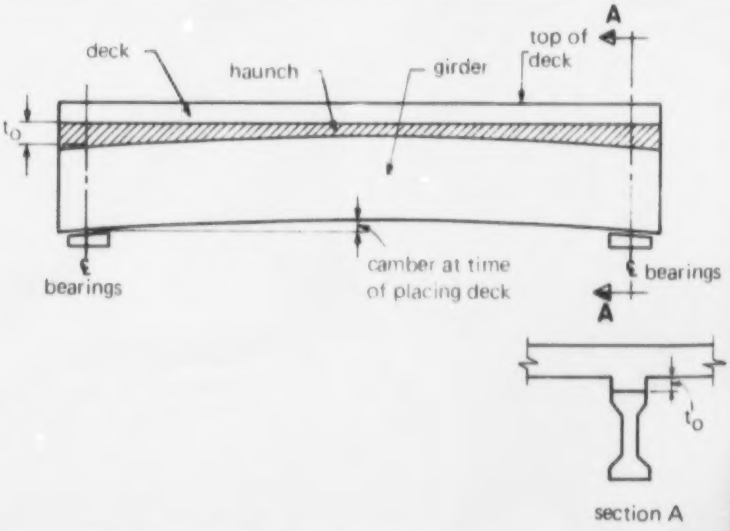
Composite Semi-Continuous Structures **C9 — 21.4**

Figure **Loading Stages Affecting Long-Term Deformations** **C9 — 20.2(a)**



(sdl) = superimposed dead load

Figure **Haunch Dimensions** **C9 — 20.2(b)**



C9 — 21.4.1 The provisions of this clause do not apply when continuity is achieved by post-tensioning.

C9 — 21.4.2 In semi-continuous structures, positive restraint moments are caused by the effects of creep, shrinkage and temperatures as shown in Figure C9 — 21.4.2.

In the past, the joints between the girder ends and diaphragms of composite semi-continuous structures with inadequate positive moment reinforcement have opened. The positive moments due to the effects of creep and shrinkage may be computed and provided for in accordance with Reference 62.

C9 — 21.4.3 The theoretical compressive stress at the end regions of the girders over continuous supports resulting from the combined effects of prestressing, superimposed dead load and live load is generally high, but does not affect the flexural resistance [62]. The compressive stress can be lowered by reducing the eccentricity of the prestressing steel and/or debonding some of the strands at the girder ends.

omposite onstruction **C9 — 22**

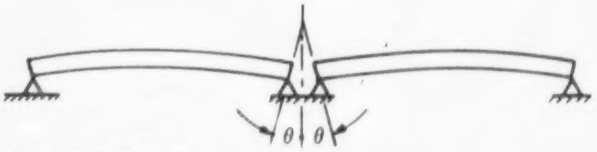
General **C9 — 22.1** The provisions of this clause have been adopted, with appropriate modifications, from References 13, 28 and 34.

Flexure **C9 — 22.2** Differential shrinkage strains induce tensile stresses in the deck slab and also in the bottoms of the girders. These stresses are significant and should be added to the stresses due to loads at serviceability limit state Type II.

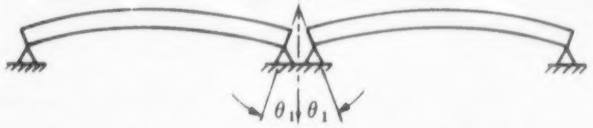
Shear **C9 — 22.3**

C9 — 22.3.2 When haunches to compensate for camber or deflection are provided, the stirrups should be anchored in the deck slab above the bottom layer of reinforcement in the slab as shown in Figure C9 — 22.3.2(a); otherwise, the haunch should be reinforced with ties anchored in the slab as shown in Figure C9 — 22.3.2(b).

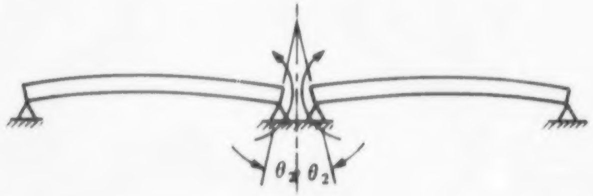
igure **Deformations and Positive Restraint Moments in a Two-Span Semi-Continuous Member** **C9 — 21.4.2**



(a) initial deformation



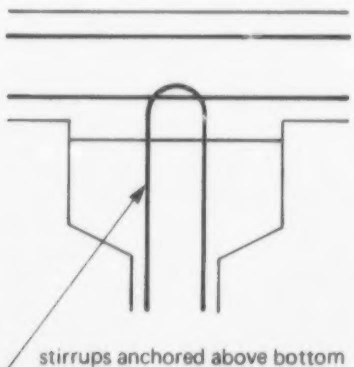
(b) final deformation if left as two simple spans



(c) final deformation and positive restraint moment if spans are made continuous after prestressing

T-Girders		**C9 — 23**	The provisions of this clause have been adopted with modifications from Reference 34.
	Isolated Girders	**C9 — 23.4**	The stipulations in Reference 34 requiring the flange thickness to be greater than 0.5 times the web width and, in monolithic construction, the flange width to be less than 15 times the web width, were found to be impractical. These limitations were changed to 0.33 and 9 times, respectively.
Box Girders		**C9 — 24**	The provisions of this clause have been adopted, with modifications, from References 13 and 34.
	Flange Thicknesses	**C9 — 24.3**	The dimensional limitations for top and bottom flanges are based on current practice in Ontario.
	Top and Bottom Flange Reinforcement	**C9 — 24.4**	The top and bottom flange reinforcement specified in this clause is a minimum requirement. The actual amount of reinforcement provided should be based on analysis in accordance with Section 3 and the relevant clauses in Sections 7 and 8.
Multi-Beam Decks		**C9 — 25**	Multi-beam decks consisting of precast units placed side-by-side should be provided with a cast-in-place structural concrete slab, since some structures with wearing surfaces of only asphalt, concrete, or both, have cracked longitudinally between the units, allowing intrusion of de-icing chemicals and resulting in structural deterioration.
Segmental Girders		**C9 — 26**	The provisions of this clause have been adopted, with modifications, from References 34, 61, 63 and 64.
	Design of Superstructure	**C9 — 26.2**	The major difference between the design of cast-in-place structures and segmental structures is in the analysis of flexural and shear stresses due to dead load. In segmental structures, stresses should be investigated at every joint between segments under all loading stages including prestressing. Dead load and prestressing moment redistribution due to creep constitutes an important design consideration in segmental construction. With cast-in-place segments, deflections should be monitored and adjusted as construction proceeds. Due to early loading of cast-in-place segments, deflections and deformations are greater than those in precast segments, where the concrete is more mature at the time of loading. Deflection computations should include dead load, prestressing, erection loads, and concrete creep and shrinkage. The size of precast segments is generally controlled by shipping and handling restrictions. Flanges and webs should be designed to resist transverse and longitudinal loads as well as to accommodate prestressing tendons. Fillets should be provided to avoid stress concentrations and to allow draping of prestressing tendons. Precast segments should be match-cast and shear keys should be used to align segments and transmit shear before the epoxy at the joint sets.

Figure **Shallow Haunch** **C9 — 22.3.2(a)**



The epoxy acts as a lubricant which assists in guiding the segments into proper alignment during placing. It also acts as a stress distribution material during erection and post-tensioning. In addition, any small cavities on the faces of the segments are filled and the epoxy serves to seal the joint. Epoxy also provides some of the tensile and shear resistance across the joint.

Details such as fillets, shear keys, web stiffeners and internal anchorage blocks should be provided as required. Openings should be provided in abutment and pier diaphragms to permit movement of men and equipment during construction and for future inspection.

Design of Substructure **C9 — 26.3**

Unbalanced cantilever moments may be provided for by temporary supports, temporary vertical prestressing and bearings over the piers as shown in Figure C9 — 26.3 or other methods [65].

The temporary supports, temporary vertical prestressing in the piers or other methods to resist unbalanced cantilever moments during construction should be proportioned to meet the requirements of Section 2.

Piles **C9 — 27**

General **C9 — 27.1**

Provisions in this clause deal with the structural aspects of prestressed concrete piles. The structural resistance of a pile rarely controls its design except in structures such as trestles. The design is usually controlled by geotechnical considerations and forces during handling and driving.

Specified Concrete Strength **C9 — 27.2**

The desirable concrete strength for piles is generally between 40 and 55 MPa in order to reduce spalling and cracking during driving.

Effective Prestress **C9 — 27.3**

The minimum effective prestress levels of 3 MPa and 5 MPa are empirical values [66]. Prestress levels below these have resulted in excessive cracking during handling and driving. When very low, or very hard driving resistance is expected the pile should be provided with a prestress of 5.5 to 6.5 MPa. The effective prestress of 0.20 f'_c is considered to be an upper limit.

Handling **C9 — 27.4**

Toggle and lifting holes cause stress concentrations and should be avoided.

Cross-Sections **C9 — 27.5**

Hollow piles should not be used in locations where water inside the piles can freeze and damage the piles.

Concrete Stress Limitations **C9 — 27.6**

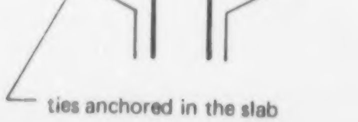
The concrete stress limitations are based on recommendations in References 66 and 67.

Factored Resistance **C9 — 27.7**

The expression for factored resistance in compression is based on References 68 and 69. The factors in the expression for factored resistance in flexure have been derived from data in Reference 68.

The factored resistance of prestressed concrete piles subject to compression and flexure may be estimated using data in Reference 66, or from the idealized interaction diagram given in Figure C9 — 27.7. Any point located inside the idealized interaction diagram represents a safe combination of flexure and axial load.

Figure **Deep Haunch** **C9 — 22.3.2(b)**



Sections Within Development Length	**C9 — 27.8**	This clause is applicable only when the pile head is subject to flexure or tension.
Reinforcement Details	**C9 — 27.9**	The additional spiral reinforcement at the heads and tips of piles provides resistance to bursting due to prestress and during driving. The requirements for spiral reinforcement are based on Reference 34. The longitudinal reinforcement should consist of a minimum of four prestressed tendons spaced equally around the pile section.
Splices	**C9 — 27.11**	This clause is adopted from Reference 70. For friction piles, the splice should not extend beyond the plan area of the pile.

Figure **Temporary Provision for Unbalanced Cantilever Moments** **C9 — 26.3**

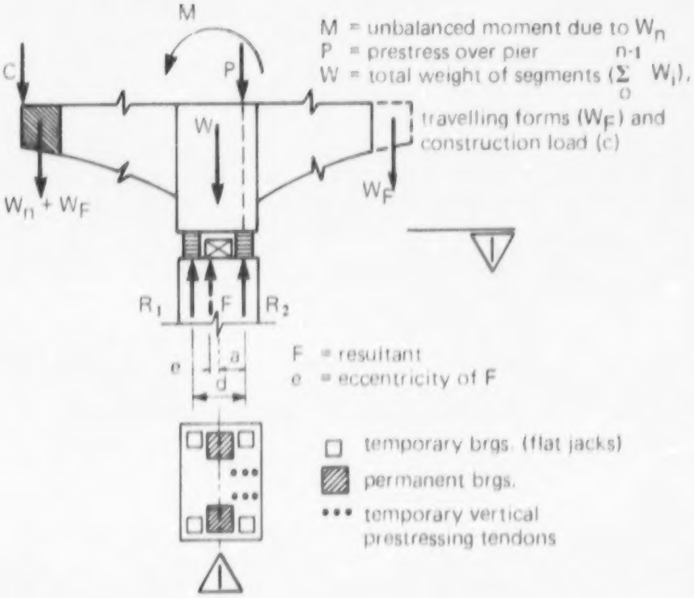
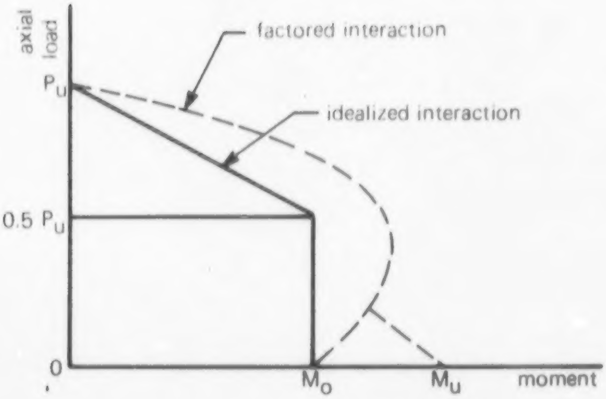

M
C
P
W
$W_n + W_F$
W_F
R_1
F
R_2
e
a
d
M = unbalanced moment due to W_n
P = prestress over pier
W = total weight of segments ($\sum_{0}^{n-1} W_i$),
travelling forms (W_F) and construction load (c)
F = resultant
e = eccentricity of F
☐ temporary brgs. (flat jacks)
▨ permanent brgs.
••• temporary vertical prestressing tendons

Figure **Interaction Diagram for Piles** **C9 — 27.7**



Poles

C9 — 28 The provisions of this clause have been adopted, with modifications, from Reference 71.

General **C9 — 28.1** The design of poles is based on empirical rules and should be confirmed by full-scale tests. The tests also serve to check the effects of apertures for electrical wiring and the manufacturing process on the resistance of poles.

Concrete Stress Limitations **C9 — 28.4** At serviceability limit states Type II loads, the tensile stress has been limited to 0.80 f_r, but it is reasonable to exceed this value under extreme wind load.

Factored Resistance **C9 — 28.5** The expression for factored resistance in flexure is the same as that for hollow circular piles in Clause 9 — 27.7.3. The expression for torsional strength has been derived from the space truss analogy [72].

Deflection **C9 — 28.6** The deflection limit is based on aesthetic considerations.

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C10 Structural Steel

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Scope

C10 — 1

Requirements are given for the design of structural steelwork for such simple and continuous span bridges as rolled beam, plate girder, truss, arch, composite beam and girder, composite box girder, hybrid girder, horizontally-curved girder, and orthotropic deck bridges including detail parts, welds, and bolts required in fabrication and erection. Those aspects of fabrication and erection having a direct bearing on design are covered. For certain types of structures, as noted in the appropriate clauses, the designer may have to meet more detailed design requirements and, in such cases, approval of the design approach must be obtained.

Materials

C10 — 4

C10 — 4.1.1

The design requirements contained in Section 10 have been developed on the assumption that the materials and products which will be used are those listed in Clause 10 — 4. These materials and products are covered by standards prepared by the Canadian Standards Association or the American Society for Testing and Materials. The use of materials and products other than those listed is permitted, but the designer should assure himself, when this option is used, that the materials and products have the characteristics required to perform satisfactorily in the structure. In particular, ductility, weldability and fracture toughness may be as important as the strength of the material. Only steels exhibiting a yield plateau followed by a strain hardening range should be used in compact sections.

C10 — 4.1.2

The three primary factors influencing the susceptibility of a structure to brittle fracture are [1]:

1/ tensile stress level and range;

2/ size of crack or severity of discontinuity that initiates a fracture;

3/ material toughness at the temperature, loading rate and plate thickness involved.

The Ontario Highway Bridge Design Code establishes loading criteria and specifies resistance functions for steel members, which together provide adequate margins of safety, combined with a reasonable degree of economy. Using a value of resistance smaller than that specified in the Code might seem to be an effective method of reducing the possibility of brittle fracture; however, this is an expensive alternative and, by itself, could be inadequate. Consequently, designers normally should concentrate on items (2) and (3) of the primary factors mentioned above. This can best be done by a fracture control plan which embodies the following features:

a) Pay attention to design details so as to minimize stress raisers and avoid restraint conditions which reduce the ductility of the member.

b) Select a type of steel that has adequate notch toughness from among the grades listed in CAN3-G40.21-M81.

c) Ensure by adequate inspection that fabrication and erection are carried out by a well-qualified steel fabricator, using approved welding procedures and appropriate electrodes to keep the size of cracks and severity of discontinuities at acceptable levels.

With respect to feature (b), considerable research has been conducted and guidance is available for the designer [1, 2, 3]. Charpy V-notch test values can be used as a guide in selecting a suitable steel grade, provided that the enormous difference in strain rate between a Charpy test and the response of bridge members to imposed loads is recognized. The strain rate experienced by Charpy test specimens is approximately 10 000 times as great as that normally experienced in bridge members. Research, using fracture mechanics procedures, has translated this difference in strain rate into a temperature shift criterion that can be used by designers in the material selection process.

The temperature shift criterion, which has been adopted by the Committee on Bridge and Structures of the American Association of State Highway and Transportation Officials, states basically that when steel toughness criteria are specified, for steels with a yield stress not exceeding 380 MPa, Charpy V-notch tests should be conducted at temperatures 40°C above the minimum expected service temperature of the bridge. For steels with a yield strength of 690 MPa, the temperature shift is reduced to 15°C, and linearly interpolated values should be used for steels with yield stresses between 380 and 690 MPa.

Accordingly, when designers specify toughness criteria for steel, in the form of Charpy V-notch tests, it is recommended that they specify a test temperature 40°C higher than the minimum expected service temperature of the bridge for CAN3-G40.21-M81 grade 260AT, 300AT, and 350AT steels, and 15°C higher than the minimum expected service temperature for grade 700QT. Minimum average absorbed energy of 20 J for grades 260AT and 300T, 27 J for grade 350AT, and 34 J for grade 700QT should provide adequate toughness.

Procedures for conducting the Charpy tests are specified in CAN3-G40.21-M81 and G40.20-M81. Although the T and A steels have the chemical composition required to attain the toughness characteristics listed in Table 10 of G40.21-M81, designers should be aware that the expected toughness can be assured only by specifying that the Charpy tests be conducted. Therefore, where such assurance is necessary, as in fracture critical members or primary tension elements where a single fracture could lead to a collapse of the structure, Charpy tests should be specified.

C10 — 4.1.3 CAN3-G40.20-M81 permits the supply of plates up to 13 mm thick from coils. These plates are originally produced in coil form, then unwrapped, sheared to size, and flattened if necessary to meet G40.20M tolerances. The resulting plate may contain residual stresses from the flattening operation that are not present in plate produced by normal rolling operations. Until more experience is gained with the use of these plates they should not be used in bridge elements, particularly those subject to compressive stresses.

Bolts

C10 — 4.4 Prior to 1974, ASTM A490 bolts were not included for use in either AASHTO or CSA highway bridge design standards. Changes in the ASTM A490 specification, and research indicating an acceptable fatigue life, resulted in adoption of A490 bolts by AASHTO in 1974 and by CSA in 1976. The *Specification* [4] and the *Guide* [5] prepared and approved by the Research Council on Riveted and Bolted Structural Joints endorse the use of ASTM A490 bolts in bridges. At the present time, however, due to the possibility of hydrogen stress cracking [5], galvanized A490 bolts should not be used.

Design Theory and Assumptions

C10 — 5

C10 — 5.1, C10 — 5.2 At serviceability limit states, satisfactory performance must be assured. The prevention of permanent deformation and slip in bolted connections must be considered at a load level likely to occur relatively few times during the life of a structure.

C10 — 5.3 The requirements at ultimate limit states generally relate to a collapse mode of the structure and the load effects, increased to account for possibly greater loads, are compared to the resistances reduced by a performance factor to account for possible under-runs in strength.

C10 — 5.5 The performance factor for computing flexural resistances of shapes fabricated from plates is specified as 0.95, based primarily on the statistical analysis of the variation of geometric and material properties of the plates and the statistics of the tested-to-predicted flexural resistances of steel members [36]. The performance factor for other member resistances at this time remains 0.90.

C10 — 5.6.1 Although the strength of a section is normally defined as the ultimate collapse, or yield strength, the only actual analysis allowed is an elastic analysis; thus, no plastic redistribution of bending moments is considered.

Design Detail Requirements

C10 — 6

C10 — 6.1 Details with high stress concentrations and details causing low resistance to brittle fracture or fatigue failure should be avoided.

Weld details which result in severe restraint in the direction through the thickness of the material should also be avoided. In such cases, alternate weld details to minimize constraint, or bolted connections, should be considered.

C10 — 6.5 Floorbeams and diaphragms at piers and abutments should be designed to allow jacking of the superstructure for replacement of bearings.

C10 — 6.7 Experience indicates that defects or discontinuities in the weld may contribute to a fatigue or fracture failure.

Tension Members

C10 — 7

C10 — 7.1 The design and behaviour of tension members is integrally related to the connection proportioning and detailing. Consequently, Clauses 10 — 7 and 10 — 16 are similarly related.

Recent research summaries and bibliographies [6, 7, 8] have been used in the development of these clauses.

The net area of a tension member connected by mechanical fasteners is computed on the basis that the hole diameter is 2 mm greater than specified by the designer. This provides for the effect of material distortion around a punched hole and/or minor dimensional changes which may occur during steel fabrication and erection, e.g., match reaming of connection parts.

C10 – 7.1.3

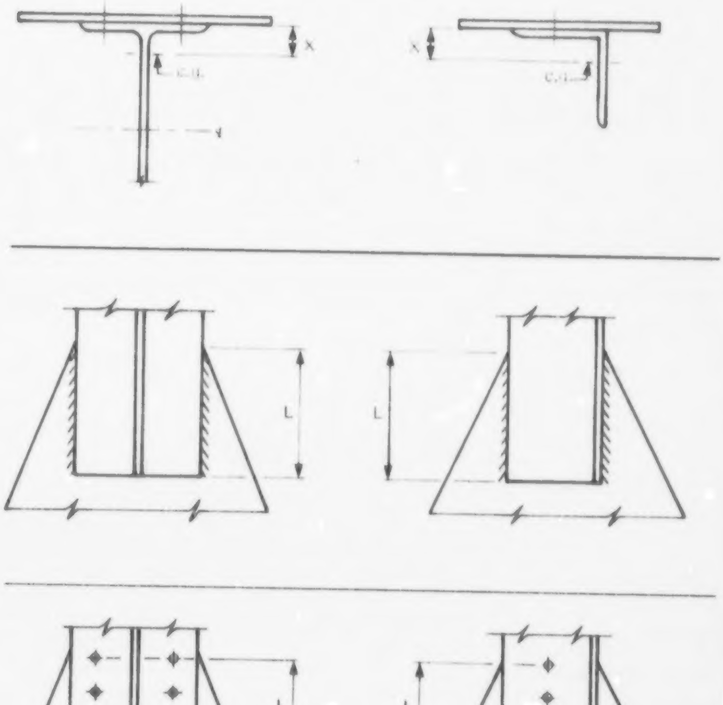
Tests and theory show that there is an upper limit to the amount of material that is effective in contributing to the tensile strength of a member regardless of the nominal net area calculated according to Clauses 10 – 7.1.1 and 10 – 7.1.2. This limit is primarily a function of the ratio of the yield strength to the ultimate strength. Upper limits on the critical net area used to compute the tensile resistance are, therefore, given as a function of the yield and the ultimate strengths.

C10 – 7.1.4

In connections where some of the elements (e.g., flanges, webs, etc.) are not directly connected with a proportionate number of bolts or welds, the tensile resistance of a member may be reduced below that computed by use of the simple net area of all elements. An improved correlation between computed tensile resistance and numerous test results is obtained by using the modified net area A'_n. The modified area accounts for the phenomenon of shear lag which precipitetes premature net section failure when the length of the connection is not adequate to develop the full net section ultimate strength. Detailing changes may also be employed to reduce the effect of shear lag. An example of such a detailing change would be the addition of a doubler plate to the web of a wide flange or I-section within a double gusset truss connection. Here, the stiffening of the web which is not directly connected provides shear stiffness and assures the development of the strength of the web at the net section.

Examples showing dimensions which should be used in calculating the net area to account for shear lag are shown in Figure C10 – 7.1.4.

Figure **Examples of Dimensions Used in Calculating Shear Lag Reduction Factor** **C10 – 7.1.4**



C10 — 7.2

The computed axial tensile resistance of a steel member connected by welds or bolts may be controlled by the yield strength or the ultimate strength of the material. The relationships presented in Clause 10 — 7.2(a) and (b) ensure that upon reaching the level of factored resistance the members may exhibit net section yielding at the ultimate limit state and will have a minimum reserve capacity of approximately 20% of the computed tensile resistance before rupture.

In the case of mechanically-fastened connections, the connection design may provide a member behaviour such that gross yielding of the member occurs before net section rupture, i.e., $A_nF_u > A_gF_y$. While this behaviour is, in general, desirable because of the reserve ductility permitting load redistribution in the event of member failure, the fact that such connection proportions may not be practically achieved is accounted for in the second expression in Clause 10 — 7.2(a). Thus, when the connection is proportioned such that member rupture at the net section is likely to occur before gross section yielding, the computed tensile resistance is reduced further in proportion to the ratio A_n/A_g.

C10 — 7.3

The interaction of tensile force and bending moment is specified as a linear relationship, which is generally conservative for material-related strengths as opposed to those involving stability. If biaxial moments are present, the moment ratio may be expressed as the sum of ratios corresponding to the respective axes.

Compression Members

C10 — 8

C10 — 8.1

Compressive resistance of steel members is expressed as a function of the gross area, the yield stress, and the slenderness ratio. The slenderness ratio is computed by dividing the effective length by the radius of gyration of the cross-sectional area. Effective length and radius of gyration are calculated assuming fully-elastic behaviour. Further information regarding the computation of effective length can be found in Reference 9. This provides a useful compilation of current research findings to support the requirements for metal compression members in this clause. Computer programs to determine the in-plane buckling capacity of truss members [10] and the out-of-plane buckling capacity of pony truss members [11] based on research conducted by the Ministry are available.

C10 — 8.2

The local buckling requirements for elements of members subjected to uniform compression as tabulated in Table 10 — 8.2.1 have been set so that any compression element will be capable of attaining the yield strength of the material. The form of the width-to-thickness requirements derives from the classic buckling strength for a long plate in uniform edge compression. Inelastic behaviour and the effect of residual stresses have been estimated and the relations have been checked and verified by numerous test results.

C10 — 8.3

The three-part compressive resistance formulation presented in Clause 10 — 8.3(a) to (c) is based on the maximum strength theory for real columns with small initial crookedness, as opposed to the classical modified Euler formulation which has been in use for many years. C_r/ϕ represents the mean strength of light- and medium-weight rolled sections and welded sections made from flame-cut plates. The effects of residual stresses and initial out-of-straightness of 1/1000 were taken into account in the derivation of the curve which is essentially Column Curve 2 proposed by the Structural Stability Research Council (SSRC).

Idealistically, for each column cross-section which can be characterized by a specific residual stress distribution and a typical maximum value of crookedness, there is a maximum strength column curve which can be set forth for the calculation of compressive resistance. The SSRC has grouped all columns into one of three categories and an empirical expression for the mean strength of all columns in a given group is recommended for the calculation of the compressive resistance of columns of the particular category.

The use of Curve 2, incorporated with minor modifications, as the most appropriate for sections in common use in Canada, will result in a conservative computed resistance for certain columns which would be classed in a category appropriate for those column types where Curve 1 is applicable. It has been shown [12] that the new curve results in a relatively uniform value of the Safety Index β over all slenderness ratios.

Caution must be taken not to use the relationships shown in Clause 10 — 8.3(a), (b) and (c) for heavy sections and welded sections fabricated from universal mill plates. Column Curve 3 of the SSRC would be appropriate for such unusual applications. It should be noted that the use of the Clause 10 — 8.3 formulae for welded sections composed of flanges which have flame-cut edges is not prohibited. In general, special attention should be given to built-up members to avoid weldments which would tend to increase the compressive residual stress in fibres remote from the centroid of the member.

C10 — 8.4 Members subject to axial compression and bending are treated either as compact or non-compact sections as defined in Clause 10 — 9. A beam-column must satisfy at least two requirements to ensure its adequacy under combined loading:

a) a strength requirement to resist moment and thrust at joints and brace points, and

b) a stability requirement to resist the axial load in the presence of primary bending moments.

For compact H-shapes, Clause 10 — 8.4.1.1(a) and (b) are strength checks, the former controlling when moments predominate. Overall member stability is checked using the empirical equation of Clause 10 — 8.4.1.1(c); C_{ex} and C_{ey} are the classic Euler buckling loads with respect to the principal buckling planes.

In computing the factored resistances of special types of built-up members, such as those with batten plates or lacings, the effect of shear should be included [1].

For non-compact sections, the relationships of Clause 10 — 8.4.1.2(a) and (b) are formulated from the extreme fibre stress limit at one corner of the section and, as in the foregoing, consist of strength and stability requirements, respectively.

The simple bilinear approximation of the moment gradient effect is set forth in Clause 10 — 8.4.2. These values of ω are greatly simplified for beam columns subjected to lateral loading; improved values may be obtained for specific cases from Reference 9, page 204.

Beams and Girders

C10 — 9

C10 — 9.1.1 The practice of proportioning beams and girders on the basis of the properties of the gross section, unless the reduction of flange area by bolt holes exceeds 15% of the gross flange area, has been used for many years and is based on the increase in member strength provided by strain hardening of the steel and the additional material provided in splices [13].

C10 — 9.1.2 Extensive research [14] has shown that beams and girders with partial length cover plates exhibit relatively poor fatigue behaviour. Although Section 11 provides design criteria for beams and girders with partial length cover plates, their use is seldom necessary and rarely economical in new construction. Accordingly, their use is not permitted except under unusual circumstances.

Compact Sections

C10 — 9.2 The slenderness of the flange and the web of a compact section is restricted in order to prevent local buckling occurring before the plastic moment of the beam or girder is attained. The limits specified in Clause 10 — 9.2.2 are based on extensive research [15, 16].

The use of factored plastic moment capacity ϕZF_y for member resistance does not mean that plastic analysis taking into account the redistribution of moment is permitted for bridges designed using this Code.

The attainment of plastic moment capacity requires that the entire cross-section reaches the yield point of the material before failure. This can be achieved with the slenderness requirements stipulated for compact sections. If plastic design techniques were to be used, more restrictive slenderness limits would be specified, since a plastic design section must not only reach its plastic moment capacity, but must also retain that capacity while a significant degree of rotation takes place permitting redistribution of bending moment in the member. This is not permitted in this Code.

The lateral bracing requirements of Clause 10 — 9.2.3 (also based on research [17]), must be adhered to in order to achieve the full plastic moment capacity. If these lateral bracing requirements are not met, a reduced moment resistance of the beam or girder, computed in accordance with Clause 10 — 9.2.4, must be used. The requirements of Clause 10 — 9.2.4 recognize that the resistance to lateral bending of an unbraced beam depends upon the weak-axis bending stiffness of the cross-section, EI_y, and its resistance to twisting. The resistance to twisting is composed of two components:

- the St. Venant torsional resistance, denoted by the term containing GJ in the formula in Clause 10 — 9.2.4, and
- the warping torsional resistance denoted by the term containing the warping constant C_w in the formula.

A more detailed explanation is available in References 18 and 33.

The shear and combined shear and moment requirements of Clauses 10—9.5 and 10—9.2.5, respectively, are based on research conducted on plate girders [19, 20], but are applicable to all beams and girders. In Clause 10—9.5, formula (a) recognizes that beams and girders with relatively thick webs can reach the shear yield stress value (i.e., the Von Mises value) of approximately 0.58 F_y. Formula (b) deals with stiffened webs in which failure occurs by inelastic buckling, with post-buckling strength in the stiffened webs, while formula (c) covers stiffened webs in which failure occurs by elastic buckling, again with post-buckling strength in the stiffened webs. References 18, 19, 20, and 21 provide more detailed information.

Non-Compact Sections

C10 — 9.3

The requirements are similar to those for compact sections except that, for non-compact sections, only the factored yield moment, ϕSF_y, can be anticipated and the formulae are adjusted accordingly. Since only the extreme fibres of the beams and girders must reach the yield point in order to attain the yield moment in the cross-section, the slenderness requirements for the flanges and web are less severe than those for a compact section [18, 22].

For non-compact sections which are not doubly symmetric, methods of analysis for computation of resistances given in Reference 79 are acceptable.

Sections with Stiffened Webs

C10 — 9.4

The plate girder requirements are based on research conducted by Basler and Thürlimann almost 20 years ago [19, 20, 23, 24]. Although more recent research [25, 26, 27] indicates that the Basler-Thürlimann approach may be somewhat conservative, particularly for girders with substantial flanges, incorporation of some of the new concepts into bridge design rules requires further development work. The Basler-Thürlimann approach has been retained in this Code for the present time. For girders with relatively thick webs, with or without stiffeners, the formulae for shear resistance and moment resistance given in Clauses 10—9.5 and 10—9.3 apply also to plate girders. For girders with slender webs, the compression flange tends to buckle into the web, and a formula for reduced moment resistance, based on the slenderness of the web, is provided. The maximum web slenderness permitted without the use of longitudinal stiffeners is based on research conducted in the United States [28, 29].

Bearing Stiffeners

C10 — 9.6

The requirements for bearing stiffeners are based on successful practice developed over a period of more than 20 years.

Intermediate Transverse Stiffeners

C10 — 9.7

The requirements for intermediate transverse stiffeners are based principally on the research work of Basler [19]. The formula for moment of inertia of the stiffeners has been in the AASHTO Specifications since the 1950s and is based on the theoretical work by Bleich [30]. Reference 18 contains more details on the development of intermediate transverse stiffener requirements.

Longitudinal Stiffeners

C10 — 9.8

The longitudinal stiffener requirements in Section10 are essentially the same as the AASHTO load factor design requirements, and are based on the recommendation of Reference 31. Generally speaking, longitudinal stiffeners should be used only in relatively deep girders. The use of more than one longitudinal stiffener on each side of the web can seldom be justified. References 26, 27, and 34 provide more information on the effectiveness of longitudinal stiffeners.

Lateral Bracing, Cross Frames and Diaphragms

C10 — 9.9

The requirements of this clause are based principally on experience.

It is noted that the differential displacement of two adjacent girders may cause greater load effects in the bracing system than those due to lateral loads. It is important to detail the connections of lateral bracing systems to minimize fatigue problems [35].

Composite Beams and Girders

C10 — 10

C10 — 10.4

The clause is provided to ensure stability and to prevent buckling of the compression flanges in the positive moment regions until concrete in the deck slab is of sufficient strength to provide adequate lateral support.

C10 — 10.5.2

The factored resistance of the compact sections in the positive moment regions is evaluated on the basis of yield stress of the steel, yield stress of the reinforcing steel, strength of the equivalent concrete stress block, and the degree of shear connection at the interface between concrete slab and steel beam of the section under consideration. The failure may occur due to yielding of steel or crushing of concrete in the slab. Appropriate performance factors are to be applied to different materials.

C10 — 10.5.2.2 The factored resistance of the composite beam varies along the length of a span. Therefore, factored compressive resistance forces for concrete slab and reinforcement, steel section, and the sum of the factored resistance of shear connectors between the section under consideration and the section of zero moment, should be determined and the least value used in computing the factored moment resistance of the section.

C10 — 10.5.2.3 This clause gives expressions for the determination of the depth of the compressive stress block and factored moment resistance of the section when the neutral axis falls within the concrete slab or is at the interface of slab and steel section.

C10 — 10.5.2.4 An expression is given for the determination of the factored moment resistance when the neutral axis falls outside the concrete slab.

C10 — 10.5.2.5 An expression is given for the determination of the factored moment resistance when the degree of shear connection is the governing factor.

C10 — 10.5.3 The factored resistance of compact sections in the negative moment regions is dependent on the yield stress of the steel section, and the amount of reinforcing steel within the effective width of the slab, assuming that a sufficient number of shear connectors are provided. It may also be affected by the conditions of lateral support provided for the compression flange.

When a sufficient number of shear connectors is provided, the reinforcing steel should be included in the computation of the plastic section modulus.

When sufficient prestressing is provided to ensure an uncracked section under loads at the serviceability limit state Type II, the concrete slab may be considered in computing the properties of the composite section for this limit state.

C10 — 10.5.3.5 The performance of composite beams when reinforcing steel is extended into the positive moment region is superior to those in which this reinforcement is not thus extended.

C10 — 10.6.1.1 For symmetrical sections when the compression portion of the web in the positive moment regions meets the compactness requirements, the section is considered capable of developing full plastic stress distribution. This can be attributed to lateral restraint provided by the concrete slab.

C10 — 10.6.1.2 This clause gives a conservative estimate of the factored moment resistance of the section. Therefore, an alternative method is permitted which takes into account a plastic stress distribution in the concrete slab.

C10 — 10.9 The performance of continuous steel beams or girders when shear connectors are provided in the negative moment regions is superior to those in which shear connectors are not so provided.

Reference 35 shows that high-strength bolts inserted through holes in the flange with a nut on either side provide adequate shear connection.

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omposite Box irders

C10 — 11 The requirements of this clause reflect North American practice, which is based on research conducted in the United States [41, 42, 43, 44], and may be considered valid for bridges of up to 75 m spans. For long-span structures, it is recommended that the designer consult the technical literature.

During construction, when the box girder is open, bracing is required to provide torsional restraints at supports, to prevent distortion of the open cross-section, and to prevent rotation and, hence, cross bending stresses as given in Reference 45. Also, during construction, if a longer zone of the bottom flange is in compression, additional stiffening of this zone may be required.

C10 — 11.7 If a single bearing is used for a box girder, it is necessary that diaphragms or cross frames be provided between the boxes at the supports.

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ybrid Girders

C10 — 12

C10 — 12.1 Various experimental and theoretical studies have demonstrated that hybrid girder behaviour is virtually identical to that of a homogeneous girder of the flange steel [46, 47]. Here, flange steel of a quenched and tempered grade was used with a mild carbon steel web to create the hybrid. While web steel will yield before the full flexural yield strength of the flange is realized, the contribution of the small yielded portion at the top and bottom of the web to the flexural stiffness is marginal. Yielding of the web material is contained by the unyielded flange material and, thus, the strength of a hybrid girder may be calculated by considering the strengths of the web and flange individually.

The design of hybrid sections can, thus, be based on the moment which causes yielding of the flange material. Hybrid sections meeting compact section properties are similarly treated. Further experimental evidence may indicate that the full plastification of the web may be assumed in computing the total flexural strength.

Limits have been placed on the total variation of the design parameters, largely to allow only proportions and combinations which have had some experimental validation. For example, while the studies [46] indicate that web yielding has little effect on web slenderness or stiffener requirements, there is not sufficient information to incorporate the post buckling strength of the web in the moment resistance formulations. Stiffened girder webs remain based on elastic buckling formulations.

C10 — 12.2 The moment resistance formulations presented here consist essentially of a reduction factor incorporated in the conventional moment resistance calculations for the various sections. The reduction factors approach is used for computational convenience.

C10 — 12.3 The shear resistance formulation is based on the assumption that only the web carries shear and limits the shear according to the elastic shear buckling.

Horizontally-Curved Girders

C10 — 13

C10 — 13.2 I-shaped girders are relatively flexible in torsion and, therefore, intermediate diaphragms or cross frames of sufficient bending capacity to resist torsional effects should be provided.

An analysis of the entire structure which takes into account the complete distribution of loads to the various members should be used [48 to 62, inclusive].

The minimum number of girders should, preferably, be more than two.

C10 — 13.3 The relationship is provided to ensure that total combined stress due to factored flexural and lateral flange bending at the compression flange tip should not exceed the yield strength of the steel.

C10 — 13.6 Diaphragms and cross frames should be more closely spaced as curvature increases. The lateral buckling strength of a curved girder segment between points of lateral bracing is dependent on the unbraced length, the curvature of the girder, and the magnitude of the lateral bending stresses at the bracing points.

To provide adequate lateral support for the flanges, the diaphragms should be attached as near to the flanges as practicable.

Transverse stiffeners at intermediate diaphragms should be welded to both flanges to assist in resisting torsional forces.

Trusses

C10 — 14 The requirements of this clause are based on the existing ASSHTO Specifications which are also reflected in CSA Standard S6 and do not represent a change from current practice.

Orthotropic Deck Bridges

C10 — 15 This clause provides general requirements for the design of steel bridges that utilize a stiffened steel plate as a deck [63 to 67, inclusive].

C10 — 15.4.1 The expression for local transverse factored moment resistance of the deck plate allows 10% increase in factored resistance; this is similar to the present AASHTO requirement.

C10 — 15.4.2 The relationship for the factored moment resistances for the combined bending of the longitudinal rib and longitudinal girder is expressed in the limit state form and is similar to the present AASHTO requirement.

Splices and Connections

C10 — 16 The design and performance of splices and connections is integrally related to the design and behaviour of the member connected. Therefore, while the connection and splice provisions are treated separately here, the overall member and structural behaviour must be considered in connection and splice proportioning and detailing. Research summaries and bibliographies [68, 69, 70] were used to develop these clauses. The design of connections, particularly the detailing, may be further limited by the provisions of Section 11.

C10 — 16.1 General guidelines for member layout, minimum and non-quantitative requirements, are listed in this introductory clause. The various requirements have developed largely from many years of experience and satisfactory engineering practice.

C10 — 16.2 The basic philosophy which has been followed in the development of resistance formulations for connections and splices is intended to ensure that the strength of a structure is controlled by the behaviour of the member and not that of the connector. Hence for bolts, the ϕ factor has been taken as 0.67 rather than 0.90.

C10 — 16.2.1

The resistance of a high-strength bolt loaded in tension is equal to the product of its ultimate strength and a stress area. The stress area magnitude lies between the gross bolt area and the area computed on the minimum thread root radius. A special expression exists for the computation of stress area but, for simplicity, it is approximated at 0.75 A_b in determining the tensile resistance.

The tensile strength of a fastener has been shown to be unaffected by the levels of pretension prescribed for installation. However, in most applications the fastener is subject to forces caused by prying of the connected parts. The designer should include an estimate of the prying force that may occur when the connection is loaded. Reference 68 summarizes various recommendations for an approximate calculation of this force. Also, research has indicated that in a connection subject to repeated loading, a bolt which has, as part of its total direct loading, a high portion attributable to prying is more susceptible to fatigue failure. Thus, prying must be minimized in connections which are to be subjected to repeated loads. Section 11 provides fatigue requirements related to design of connections.

C10 — 16.2.2

Two limit states are considered for bolts subjected to shear, one for serviceability and one for ultimate strength. The serviceability limit state provides for a low probability of slip of the connected parts. The ultimate limit state requirements, in combination with the provisions of Clause 10 — 7, provide a member resistance which will control the ultimate behaviour of the assembly.

C10 — 16.2.2.1

The shear or slip resistance for serviceability is a function of the surface condition of the connected parts, the number of shear or slip planes, and the normal force provided by properly installed high-strength bolts. Slip coefficients vary considerably for a given steel surface condition such as "tightly adhering clean mill scale"; however, much experimental evidence collected over many years has provided sufficient statistical information to characterize the variability one could anticipate when specifying such a surface condition. Also, a specific grade of high-strength bolt installed by the turn-of-nut method will achieve a tension which is dependent on the actual fastener strength and other parameters which change from one installation to the next. Thus, the variability of fastener tension can also be characterized by a distribution obtained experimentally. The values of μ which are presented in Table 10 — 16.2.2.1 are numbers which have been calculated using the probalistic data on fastener tension and slip coefficients [9] so that the calculated resistance V_s in Clause 10 — 16.2.2.1 is an estimate of the 95% statistical tolerance limit made with 90% confidence. This limit is also referred to as the 5% probability of slip. The equation for the resistance V_s was formulated according to the data published in Reference 9 and, as a result, the quantity μ does not represent a single physical parameter such as a slip coefficient but rather accounts for the variability of the slip coefficient and fastener tension.

Continuing research and standardization of test methods will allow the future expansion of this table to include additional surface treatments. The requirement of slip resistance in all bridge splices and connections is recognition of the improved fatigue behaviour of material in such connections and the structural performance of rigidly-connected members. In many (but certainly not all) designs, this serviceability limit state will be the controlling factor in the design process.

C10 — 16.2.2.2

The shear resistance at the ultimate limit state is controlled by the shear strength of the bolt or the bearing strength of the bolted material. The latter of these failure types is defined as either the excessive deformation piling up of material ahead of the loaded bolt or the tearing out or splitting of the material. The shear strength of the bolt is realized by severing the bolt at one or more shear planes.

Extensive testing has established that the shear strength of high-strength bolts is approximately 60% of the tensile strength of the bolt material; a further 30% reduction is required if failure may occur through the threaded portion of the bolt shank. A second reduction of this bolt shear resistance occurs with increasing joint length, due to a non-uniform distribution of load among the bolts. Rather than base the unit resistance for all connection lengths on a long joint, an approximate step relationship provides for a 20% reduction for connection lengths in excess of 1.3 m.

The formulation of the two-part bearing resistance B_r was derived from a model for tear-out of the end fastener [70] and an observed limit, above which a more conventional bearing-type failure, corresponding to excessive hole elongation and piling up of material ahead of the hole, predominates.

The experimental verification of this model consisted of tensile tests of simple tension splices using one, two, and three bolts in a line. Thus, application of the bearing strength formulation to other than simple tension splices must acknowledge the possibility of tear-out mechanisms other than the one used on the basic formulation. A simple double-angle beam header connection is one such application. Current findings show that a limit of $3\phi_b$ tnd F_u on B_r is safe for this connection type; a more complex modelling of the failure mechanism limits may be less conservative.

C10 — 16.2.3 In parallel with the previous requirements for either tension or shear resistance, the expressions here are interaction relationships between tension and shear for the serviceability and ultimate strength limit states, respectively.

C10 — 16.2.3.1 For serviceability limit state Type II, a linear interaction between T_r in Clause 10 — 16.2.1 and V_s in Clause 10 — 16.2.2.1 has been assumed.

C10 — 16.2.3.2 Extensive testing of single fasteners under various ratios of combined shear and tension indicated that an elliptical curve would fit the data well and, thus, such a curve has been formulated for the resistance function interaction.

C10 — 16.3 The provisions for strength of structural steel weldments conform to CSA Standard W59 with clauses presented here to allow strength design of weldments. Particular consideration is given to the shear resistance of complete and partial penetration groove welds and fillet welds.

The resistances of welds in other categories are taken to be the same as those for the base metal. These other categories include tension or compression parallel to the axis of complete or partial penetration groove welds and of fillet welds, tension or compression normal to the throat of complete penetration groove welds and compression normal to the throat of partial penetration groove welds.

Tension other than parallel to the axis of partial penetration groove welds is considered to be the same as shear. The shear resistance of a weld must be evaluated on the basis of both the resistance of the weld itself and of the base metal adjacent to the weld.

The resistance of the base metal is given as $V_r = 0.66\,\phi\,A_m F_y$ which is consistent with that given for the shear resistance of a flexural member with a stocky web. The area of metal A_m to be used here is the area of the fusion face. The shear yield of steel is customarily taken to be $F_y/\sqrt{3}$, that is, $0.58\,F_y$. The increase between this value and that given in the equation above, $0.66\,F_y$, is attributable to the benefical effects of strain-hardening. The value of ϕ to be used in evaluating the shear resistance of the base metal will normally be taken as 0.90.

The strength of the weld metal is given as $V_r = 0.50\phi A_w X_u$. The term A_w is the effective throat area of the weld and X_u is the ultimate tensile strength of the electrode as given by the electrode classification number.

As previously stated, both the weld and the connected material are being considered in this clause. To avoid confusion, therefore, the value of ϕ to be used in calculating the weld resistance is taken as 0.90. However, as noted when discussing high-strength bolts, it is desirable to ensure that the fasteners will not fail before the connected member fails. In that case, ϕ was modified to 0.67. In the equation for fastener resistance given in this clause, the reduction is incorporated into the modifier, 0.50. In addition, the modifier relates the shear strength of the weld to the electrode tensile strength as obtained from tests.

Connection and Splice Details

C10 — 16.4

C10 — 16.4.1 The assembly of bolted parts must ensure complete contact of the parts in the vicinity of the bolt holes. Any inclusion of softer material such as gaskets defeats proper bolt installation and long-term pre-load retension. Solid fit-up of bare steel parts allows normal painting procedures to seal the joint interior from continuing atmospheric corrosion.

C10 — 16.4.2 Enlarged or slotted holes provide for convenience in more liberal fit-up tolerances in field connections but research (68) has shown that special precautions must be taken in detailing when the bolt holes exceed the normal dimension, which is 2 mm greater than bolt diameter. The limitations place an upperbound on the size of the holes which research has shown not to adversely affect the slip or ultimate performance. Larger holes may be used if the reduced slip performance is taken into account.

C10 — 16.4.3 The presence of coatings on the contacting surfaces of high-strength bolted steel will, in general, change the slip behaviour of the assembly. Only those coatings that have been tested extensively are listed.

C10 — 16.4.4 Minimum pitch and edge distance requirements have been established largely from experience and practicality of fastener installation. The model used for bearing strength calculation as given in Clause 10 — 16.2.2.2 will often suggest a minimum edge distance of 1.5 times bolt diameter in the line of loading, and members having only one or two bolts in the line of load have this edge distance as an absolute limit. Maximum edge distance requirements similarly have been developed from experience and, also, from the practical requirements of sealing the interior of the joint.

C10 – 16.5.2

Moment connections which cause a tensile or compressive force to be transmitted from a beam flange to a column are proportioned by assuming that the transmitted force is closely approximated by the factored moment divided by the depth of the beam and that the resistance of the column web includes the resistance of stiffeners when they are present. The first expression, B_r, for the compressive resistance for compact H-columns, and the expression T_r for the tensile resistance have been shown experimentally [71] to be adequate to ensure the development of the plastic capacity of the framing beam. The second expression, B_r, for non-compact H-columns, is conservatively taken as 90% of the elastic buckling strength of a plate with both edges simply supported.

Pins, Rollers, Rockers, and Bearing Plates

C10 – 17

C10 – 17.1

The bearing resistance given for machined, accurately sawn or fitted parts in contact reflects the fact that a triaxial compressive stress state tends to develop which restricts yielding of the parts in contact. The values given are based on the previous working stress design standards which experience has shown to give satisfactory results. A lower bearing resistance is given in Clause 10 – 17.1(b) when pins are subject to rotation or likely to be removed and replaced because for these cases inelastic deformation could be detrimental.

For a cylindrical roller or rocker in contact with a flat surface, the maximum shearing stress developed due to a line load of q kips/inch is

$$\tau_{max} = 0.27\sqrt{\frac{qE}{\pi D(1-\nu^2)}}$$

as given in Reference 73, where ν is Poisson's ratio. Then the bearing resistance is

$$\frac{B_r}{\phi} = qL = \frac{\pi DL(1-\nu^2)(\tau\,max)^2}{0.27^2E}$$

The Hertz solution, as reported in Reference 74, gives the allowable load as

$$2.86DL\frac{(2.7F_y)^2}{E} = 0.0007DLF_y^2$$

The value of 0.00013 given in the Code is obtained from an empirical value of 0.0009 which itself is somewhat conservative.

Torsion Members

C10 – 18

C10 – 18.1

The expression $Q_r = 1.15\,F_y A't$ for a member of closed cross-section is derived from the sand heap extension of the soap film analogy. In the latter analogy the torsional resistance of a member is equivalent to twice the volume under a soap film over an opening of the same dimensions as the cross-section under torsion where the maximum slope of the soap film is analogous to the maximum shearing stress on the cross-section. For a closed cross-section, the soap film is flattened out over the enclosed opening.

For the fully plastic condition, the shearing yield stress $F_y/\sqrt{3}$ exists at all points of the cross-section. Thus, the slope of the soap film becomes constant and the film may be treated as a sand heap with slope equal to $F_y/\sqrt{3}$. The maximum height of the sand heap is then conservatively taken as $(F_y/\sqrt{3})t$ where t is the minimum wall thickness. Thus, twice volume of the sand heap, representing the torsional resistance, is equal to $2(F_y/\sqrt{3})A't = 1.15F_yA't$, where A' is the area enclosed by the median line of the wall.

Width-thickness ratios must be limited to allow the attainment of yield under factored loads. Reference 77 presents equations for the critical buckling stress of flat plates under pure shear and combined compression and shear. The limitation of $1100/\sqrt{F_y}$ will allow plates of all aspect ratios to reach the shearing yield stress in a state of pure shear. For plates under combined compression and shear, see Reference 77. For circular hollow section, the D/t limit specified is conservatively given for compact sections in compression.

The requirement of Clause 10 — 18.2.3 would restrict maximum stresses, for the case when normal stresses and shear stresses reach their maximum values simultaneously, to about 70% of the yield stress in tension and shear, respectively.

C10 — 18.3 For any relatively narrow element, either straight or curved, of length b, twice the volume of the sand heap, neglecting the small half-pyramids at either end, with side slope τ_y and with the height of the heap equal to $\tau_y t/2$ is:

$$2\,[1/4\tau_y t\ bt] = \frac{F_y\,bt^2}{2\sqrt{3}}$$

Therefore, the torsional resistance for the sum of all such elements is:

$$\Sigma 0.29\,\phi\,F_y bt^2$$

The above expression conservatively neglects the heap of one element sitting on the shoulder of the next such as at a web flange junction. However, if the ratio b/t is small, the area of the pyramid is only two-thirds that given above; by restricting the expression to b/t values greater than 7, the error introduced is less than 5%.

C10 — 18.4 Doubly-symmetric I-shaped sections, when the flanges are restrained, develop warping torsional resistance due to bending of the flanges as well as the St. Venant torsional resistance previously described. It is assumed that lateral shears V', acting in opposite directions to form a couple, are developed in the flanges at a point of no warping restraint. The corresponding factored torsional resistance is $Q_{r2} = \phi V'd'$ where $d' = d - t$ is the lever arm between the two shears V'. If the length over which the restraint is developed is ℓ, then the shear V' gives rise to a moment $V'\ell$ which must be developed by the fully-plastic lateral bending of the flange. This gives $V'\ell = F_y tb^2/4\ell$ from which:

$$Q_{r2} = \phi V'd' = \phi F_y tb^2 d'/4\ell$$

C10 — 18.4.2 It has been shown in Reference 76 that the combined bending and torsion capacity of I-shaped sections can be conservatively given by an interaction equation of the form:

$$\left(\frac{M_f}{M_r}\right)^2 + \left(\frac{Q_f}{Q_r}\right)^2 = 1$$

The experiments reported in Reference 76 related to fixed-ended beams in which maximum bending moment, torsional moment, and transverse shear existed at the same section.

Standardized Truss System

C10 — 19

C10 — 19.1 Standardized truss systems for various span lengths can be designed by using tables. Such tables should be established in accordance with the provisions of Clause 10 — 19.

C10 — 19.1.1 Usually the panels of standardized truss systems consist of triangulated welded frames comprising hot-rolled steel sections of open or tubular shape. To form plane trusses the panels are connected together by pins driven into eye holes of adjoining panels. Two or more such panel trusses are connected laterally or vertically and stabilized by beams, struts, bars, or welded bracing frames, and joined together by special pins or bolts.

C10 — 19.1.2 Tolerances of assembly have a cumulative effect on deflections and may influence the capacity. The effect of tolerances on the capacity of the total truss system may be assessed by load testing in the field.

C10 — 19.3 If materials are used which are produced to foreign material standards, special care must be taken to select grades equivalent to CSA Standards. In this regard, the particular importance of notch toughness must be borne in mind.

C10 — 19.4 As components of standardized truss systems are intended for permanent repetitive usage, fatigue considerations should be investigated.

Piles

C10 — 20

C10 — 20.2 An analysis of the effect of the restraint offered by the pile cap and superstructure as well as that due to the soil is required to determine the effective length of a pile.

C10 — 20.4 The compressive resistance of composite tube piles is conservatively estimated in accordance with the provisions of CAN3-S16.1-M78, *Steel Structures for Buildings — Limit States Design*.

Fabrication, Transportation, Erection, and Inspection

C10 — 21

C10 — 21.1 This clause gives requirements for the fabrication, transportation, erection and inspection of the structural steel components of bridges to ensure that the design is properly implemented.

The owner will frequently have detailed specifications covering each of the above subjects. When such detailed specifications do not exist (and unless otherwise specifically stated in the contract documents), Clause 10 — 21 is to be followed. Clause 10 — 21 includes by reference CSA Standard S16.1M, *Steel Structures for Buildings — Limit States Design*, CSA Standard W59, *Welded Steel Construction*, and CSA Standard W47.1, *Certification of Companies for Fusion Welding of Steel Structures*.

C10 — 21.5

High-strength bolt installation and inspection procedures are well defined in CAN3.S16.1-M78 and thus it is used for direct reference. A limitation is placed on the method of installation because the design values for slip resistance presented in Clause 10 — 16.2.2.1 were predicated on the turn-of-nut method. If any other means of tension control is approved for use, an adjusted design value for μ may be required.

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C11 Structural Fatigue and Fracture Control

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Structural Steel Components, Connections, and Fasteners C11—4

Stress Categories C11—4.2

A substantial amount of experimental data has been developed on steel beams since 1967 under the sponsorship of the National Co-operative Highway Research Program (Project 12-7) of the USA, which has shown that the most important factors governing fatigue resistance are the stress range and the type of detail [7, 8]. Since stress range rather than maximum stress is the most important criterion, only live load and impact stresses need be considered when designing for fatigue. Only tension or reversal stress cycles need be considered, as discussed in greater detail later in this commentary. Where it is convenient, the combined stresses from permanent and truck loading can be used; otherwise truck loading alone can be used to determine the stress range. These findings were observed to be applicable to every beam and detail examined. Beam specimens were used to overcome some of the limitations of smaller simulated specimens. These beam tests and other work available in the literature were used to develop a comprehensive specification based on stress range alone.

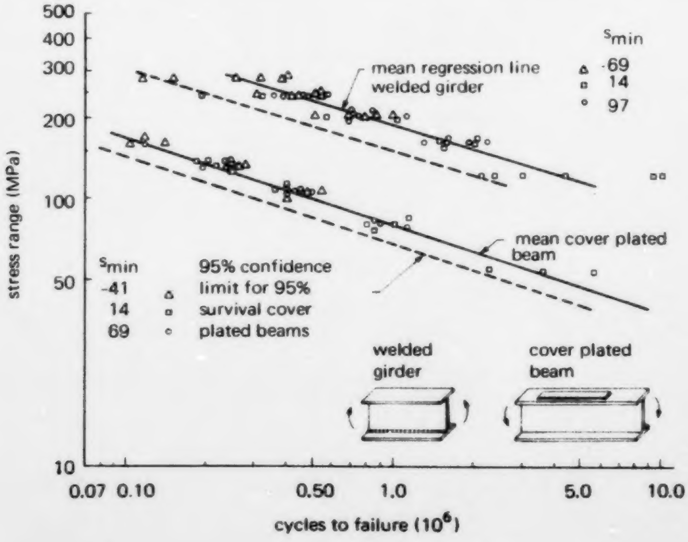
A brief summary of some of the test data is given here to demonstrate that stress range and type of detail are the two factors which are most likely to govern the fatigue strength.

Test data for a welded plate girder without attachments and a cover-plated beam are summarized in Figure C11—4.1. Stress range is plotted as a function of cycle life for several different levels of minimum stress on a log-log scale. It is visually apparent that stress range determined the fatigue life for both structural details, i.e., minimum or maximum stress did not have a significant influence on the fatigue behaviour. The ratio of minimum to maximum stress, R, did not affect the stress range/cycle-life relationship. The cover-plated beam results included wide cover plates, thick cover plates, and cover plates on both rolled and welded beams.

No significant difference attributable to the type of steel was observed for either the welded girder or cover-plated beam when a given detail was subjected to the same stress range. The data show clearly that stress range is the critical stress variable for all structural steels. The results also confirm the significance of the type of detail. The cover-plated beam provided only about 45% of the fatigue strength of the welded plate girder without attachments.

Studies on other details (referred to as stress categories in Table 11—4.2) have also confirmed that stress range alone is the only significant factor in designing a given

Figure Effect of Minimum Stress on Stress Range—Cycle Life Relationship C11—4.1



detail against fatigue. Results on beams with transverse stiffeners, attachments, and transverse groove welds have also demonstrated that minimum stress and type of steel are not critical factors [7, 8].

Groove-welded splices at flange-width transitions in A514 steel were more severely affected by the straight tapered transition. This led to the requirement for a curved radius transition for G40.21-700Q and 700QT steels shown in Figure 11—4.1, Example 11.

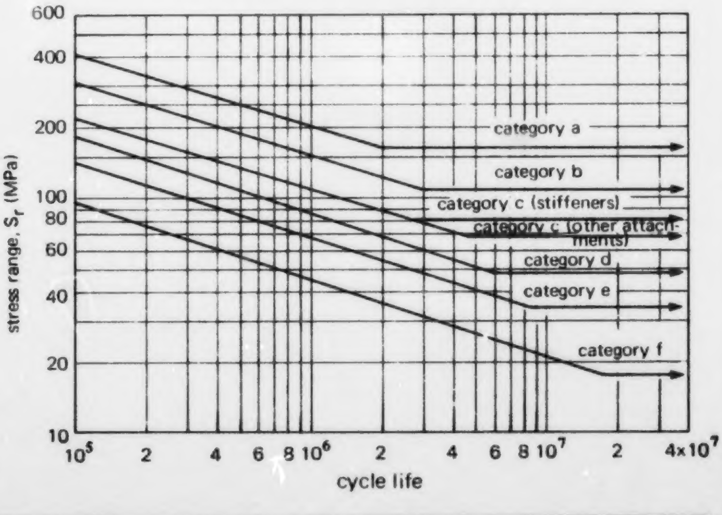
In a transverse groove weld with the reinforcement left in place, the stress concentration at the weld toe, with its associated small micro-discontinuities, is usually more critical to fatigue crack growth than nominal internal discontinuities. However, if lack of penetration, slag inclusions, or other internal discontinuities are large in size, crack growth will become more critical at the internal location [1, 20, 31]. In bridge construction, transverse groove welds that are subjected to tension or reversal of stress are nondestructively tested to prevent large internal discontinuities from occurring. Also, when the weld reinforcement is removed, so that the weld toe is not critical, a high fatigue strength results, Category b.

All evidence indicates that the termination of groove and fillet welds provides a more critical crack growth condition than internal discontinuities in the weld. The welded unit with the highest fatigue strength is the welded beam without attachments. The same strength was observed in groove-welded flange splices [7]. In flange splice details, cracks normally grow from internal discontinuities that are perpendicular to the applied stresses. When the attachments are short, as with transverse stiffeners, the fatigue strength approaches that of a comparable welded plate girder [8]. Attachments longer than 100 mm quickly approach the lower bound condition given by the cover-plated beam.

The stress range category values given in Table 11—4.1 were derived from test data using the 95% confidence limits for 95% survival. Rolled beams were used for Category a, welded plate girders for Category b, stiffeners and short 50 mm attachments for Category c, 100 mm attachments for Category d, and cover-plated beams for Category e. The stress range/cycle-life relationships are plotted in Figure C11—4.2 for each design category. After 2 000 000 cycles, the stress range approaches the crack growth threshold level for the various details and becomes a constant value. For more than 2 000 000 cycles, the fact that transverse stiffeners are less severe than 50 mm attachments is reflected by an allowable stress range of 83 MPa, which appears to be representative of the threshold level for this design condition.

Studies on full-scale welded bridge details were completed in 1976 and indicated that full-sized cover-plated beams have less fatigue strength than is implied by Category e [23]. A comparison of these test data with the results of studies on sev-

Figure **Design Stress Range Curves for Steel Components and Connections** **C11—4.2**



eral bridges which have experienced fatigue cracking shows reasonable agreement with the laboratory findings and field experience [27]. Work completed on NCHRP Project 12-15 has provided a more comprehensive data base on full-scale beams; this is the basis for design Category f [12].

The allowable stress range values for Category w were based on an examination of available test data [33, 34]. Most of the data for this class of detail were acquired many years ago [33] and also form the basis of the 1969 AISC Specification for welds in shear. These data are plotted in Figure C11 — 4.3 and show the extreme scatter in test data for welds in shear. Also plotted in Figure C11 — 4.3 are the design curves for weld shear (AASHTO and AISC Category F) and the design curve for stud shear connectors [28]. It is readily apparent that the fatigue strength is about the same for these two design conditions. Category w is used for both conditions in the present Code.

Tests on high-strength bolt connections were evaluated in Reference 9. This examination showed that Category b, which was derived from tests on welded girders without attachments, provided a reasonable lower bound estimate for the stress range relationship of bolted joints. Friction-type joints were shown to be dependent on the gross section area whereas other bolted joints were dependent on the net area. These same criteria were used for Section 11. No cyclic failures were experienced by bolts in shear, hence no special fatigue provisions are required other than the applicable strength provisions of Section 10.

Fatigue tests on anchor bolts have shown that the fatigue strength is reasonably described by stress Category e for commonly used steels, diameters, and thread types [38]. The stress range was determined using the stress area which is defined as:

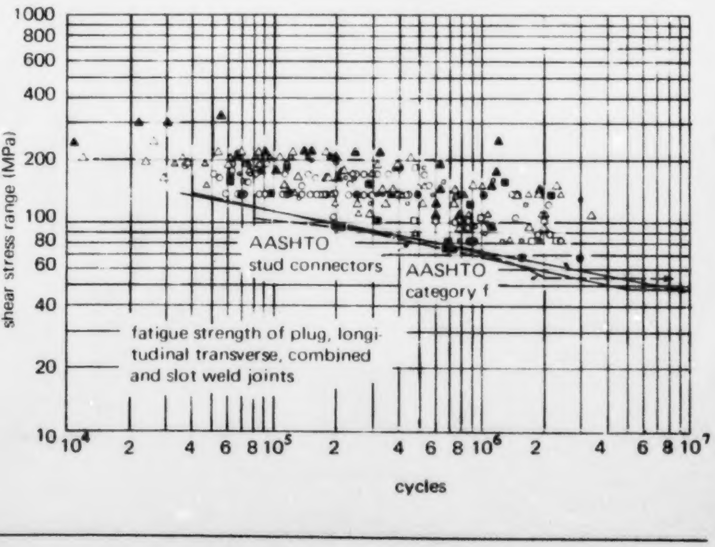
$$\text{Stress Area (mm}^2) = \frac{\pi}{4}\left(D - \frac{0.9743}{n}\right)^2$$

where: D = bolt diameter in mm, and
n = the number of threads per mm

All welding processes result in high tensile residual stresses, which are at or near the yield point in the weldment and base metal adjacent to it. These occur as the weld shrinks during cooling. Thus, the initial stages of fatigue crack growth in as-welded structures occur in regions of high tensile residual stress. Experimental observation and fracture mechanics models both indicate that much of fatigue life is consumed during these stages of crack growth. Under cyclic loading, the material at or near the initial discontinuity will be subjected to tensile cyclic stress, even in cases of stress reversal. This is the major reason why stress range alone is the variable describing the fatigue behaviour of welded joints of bridge members. As a result, the computed stress ratio R does not play a significant role in determining the fatigue strength of welded details.

Figure **Summary of Data on Welds in Shear** **C11 — 4.3**

type of weld	stress ratio			
	R=1	R=0	R=½	R=⅓
plug	△	○	□	◇
longitudinal	△	○	□	◇
transverse	▲	●	■	
combined	△	○	□	
slot	△	○	□	



An examination of the available data has shown that cracks have grown in the tensile residual stress zones of beam flanges subjected to cyclic compression alone [7, 10, 13]. However, these studies also demonstrated that the cracks stopped when they reached the adjacent compressive residual stress regions. No beams lost their load-carrying capability as a result of the compression flange cracks unless out-of-plane bending stresses were introduced.

Sections with small fatigue cracks confined to the tensile residual stress regions and subjected only to compression stress due to dead and live load are analogous to the compression splice with splice plates proportioned to carry only part of the member's strength.

As a result of this behaviour, the fatigue stress range design criterion is limited to regions subjected to tension or stress reversal. If the member is subjected to stress reversal, fatigue must be considered, no matter how small the tension component of stress range is, since the crack generated in a tensile residual stress zone could be propagated to failure even when the tension portion of the stress cycle is very small.

The most widely used method to account for cumulative damage is the Miner's Hypothesis [19]. Variable stress cycle damage is accumulated in proportion to the relative frequency of occurrence of each level of stress range. Other methods have been proposed but Miner's Hypothesis is among the simplest.

In order to evaluate the significance of random variable stress cycles and assess the applicability of cumulative damage criteria such as Miner's Rule or the RMS (root-mean-square) procedure, a program of study was undertaken in 1971 under the sponsorship of the National Cooperative Highway Research Program, Project 12-12[26]. The study was carried out at the Research Laboratory of the U.S. Steel Corporation on beams identical in geometry to those tested on Project 12-7 at Lehigh University [7].

The results of the Project 12-12 study indicated that Miner's linear damage hypothesis and the RMS stress range both provide a means of relating random variable stress cycles to constant cycle data [26]. An effective stress range can be developed using Miner's linear fatigue damage relationship $\Sigma n_i/N_i = 1$ together with Equation of S-N Curve:

$$S_{rMINER} = \{\Sigma \gamma_i S_n{}^3\}^{1/3}$$

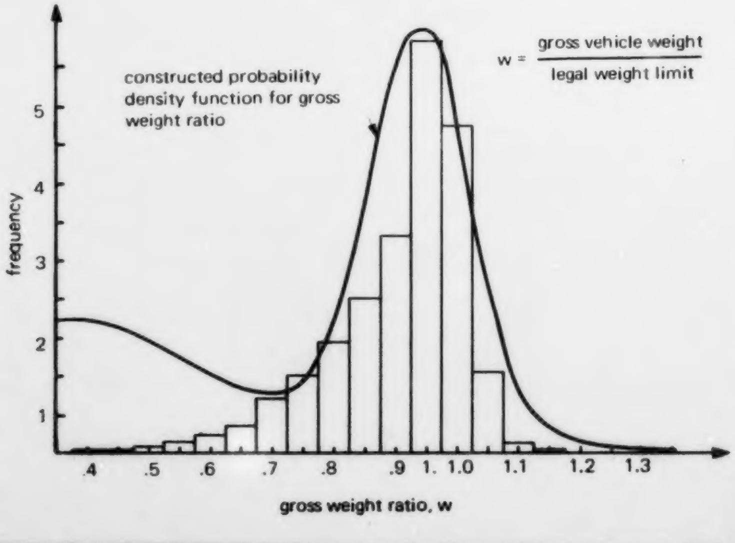
where γ_i is the relative frequency of occurrence of stress range S_n.

The RMS stress range for a variable stress spectrum is defined as:

$$S_{rRMS} = \{\Sigma \gamma_i S_n{}^2\}^{1/2}$$

Several tests were conducted with an effective stress range below the constant cycle fatigue limit (i.e., the endurance limit). Some cycles in the stress spectrum exceeded the constant cycle fatigue limit and this apparently caused all stress

Figure **Histogram of Gross Weight Ratio for the 1975 Sample Filtered by an Influence Line** **C11—4.4**



cycles to contribute to fatigue damage. Hence, if no crack growth can be tolerated and extreme life is required, all stress cycles should be less than the fatigue limit.

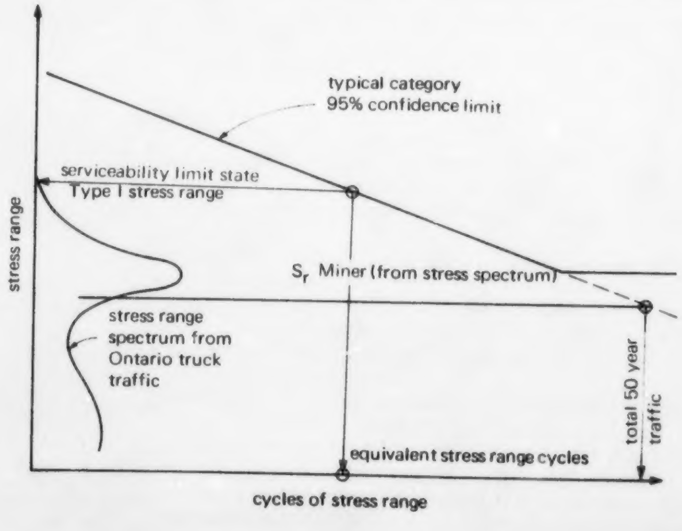
The number of cycles of maximum stress range was determined from available studies of Ontario truck traffic [2, 6]. A sample of 6375 trucks was weighed and measured [2]. Using this sample, the University of Western Ontario constructed a probability density function by passing a selected sample over several influence lines. This resulted in a characteristic histogram shown in Figure C11 — 4.4. The weight ratio, w, was defined as the ratio of the gross weight to the legal weight limit. In addition, the load spectrum was also checked against the response of the applicable configuration of a single OHBD Truck for several spans between 10 to 100 m. The truck sample used represented the upper 35% of the total vehicle population. When Miner's Rule is used to account for the cumulative damage, the variable spectrum yields [11, 26]:

$$\text{Damage} = \frac{n}{N} = \frac{\alpha^3}{N_D}(\text{ADTT})(D_L)\,\Sigma\gamma_i\,w_i^3 = 1$$

Where N_D = number of cycles of design stress range occurrences; $\Sigma\gamma_i\,w_i^3$ = constant $\approx$ 0.325 for spans less than 20 m, which represents the spectrum of vehicles travelling the King's Highway system in Ontario. For longer spans $\Sigma\gamma_i w_i^3$ tends to decrease and an adjustment factor of 0.95 was introduced to account for the decrease; ADTT × D_L = total truck volume and number of stress cycles for design life of 50 years; α = adjustment factor which is a function of accuracy of analysis, headway or multiple presence, impact and other indeterminate factors. Measurements on bridges in Canada [4, 16] and in the United States [11, 36] have demonstrated that α is always less than unity. A value of 0.7 was selected for α. Comparable criteria were developed using Miner's Rule and the RMS method. Figure C11 — 4.5 shows schematically the relationship between the equivalent constant stress range cycles of the serviceability limit state Type I loading and the load spectrum for Ontario truck traffic. The selected number of constant stress range cycles corresponds to an equivalent amount of fatigue damage.

Bridges on Class A highways are subjected to very high volumes of traffic, with the average daily truck traffic (ADTT) equal to or greater than 4000 in the busiest lane, resulting in an equivalent constant cycle design life that corresponds to the fatigue limit. The frequency distribution of the variable stress cycles results in the maximum stress range under actual traffic being about equal to the allowable stress range. As a result, the design requirements for Class A highways are applicable to larger numbers of stress cycles since nearly all stress cycles are below the constant cycle fatigue limit.

Figure **Schematic Showing Equivalent Damage of Type I Design Truck and Actual Traffic** **C11 — 4.5**



Fatigue design requirements for bridges on Class B highways are applicable to bridges subjected to between 250 and 1000 ADTT. This corresponds to 2 000 000 equivalent constant stress range cycles of the serviceability limit state Type I loading at the higher ADTT. An examination of Figure C11 — 4.2 shows that only the more severe details represented by Categories d, e and f are affected significantly when a higher volume of ADTT occurs. Assuming the frequency distribution remains unchanged, and that $\alpha = 0.7$, the number of constant stress cycles for any arbitrary frequency is $N_D \approx 2000$ ADTT. This relationship is applicable between 10^5 and the fatigue limits which provide the design condition for bridges on Class A highways.

Bridges on Class C highways are subjected to much less truck traffic because of their location and function. Class C-1 is applicable up to 250 ADTT and Class C-2 is applicable for up to 50 ADTT. Other volumes of traffic can be accounted for by means of the function N_D. Estimates of stress range can be obtained from Figure C11 — 4.2 for the desired number of equivalent design stress cycles.

High-Strength Bolts Subjected to Tensile Cycle Loads — C11 — 4.3

The provisions for high-strength bolts subjected to tensile cycle loads were developed by the Research Council on Riveted and Bolted Structural Joints [22]. These provisions are based on the limited test data that are summarized in Chapter 17 of Reference 9. The experimental data have demonstrated the desirability of minimizing any prying action due to distortion of the connected parts when high-strength bolts are subjected to cyclic loading. Various methods are available to estimate the magnitude of the prying force. Several are discussed in Reference 9. Comparison of predicted prying forces with existing experimental data are given in the same reference. These comparisons suggest that any of the available models provide reasonable estimates of the prying conditions and can be used to satisfy the requirement of Clause 11 — 4.3. It is essential that high-strength bolts be properly tightened to the prescribed preload level so that the actual stress range in the bolt is minimized.

Fracture Critical Members — C11 — 4.4

Concern regarding fracture critical members (for example, two plate girders on a simple span, ties of a tied arch, etc.), where failure of a single element could cause collapse of the structure has led to a much more conservative fatigue design requirement in order that the possibility of fatigue crack growth may be further minimized. This has been accomplished by decreasing the permissible stresses in Table 11 — 4.1 by 25%. AASHTO provides a comparable approach with a permissible stress range table for nonredundant members. The AASHTO table was developed by shifting one range of loading cycles for fatigue design, which results in reductions in allowable stress range varying from 20% to 40% [11].

Available fatigue studies have demonstrated that the strain rate does not affect the fatigue resistance [7, 8, 23]. The same conclusion has also been verified by basic crack growth rate measurements [37]. Strain rate does, however, influence fracture resistance. This fact is recognized in the AASHTO Specification and in Section 10 of the present Code. The strain rate shift is discussed in more detail in Clause C11 — 5.

Fracture Control — C11 — 5

The safety and reliability of a bridge is governed by material properties, design, fabrication, inspection, erection, and usage. The following have contributed to bridge failures by cracking:

- inadequate inspection and nondestructive testing,
- design details resulting in notches and in joints difficult to weld and inspect,
- hydrogen-induced cracks,
- improper fabrication, welding, and weld repair, and
- lack of toughness.

Excessive attention to a single preceding item will not overcome the effects of a deficiency in one of the other items. Fracture critical members require additional consideration. This includes increased material toughness and the more rigorous fatigue requirements specified in Clause 11 — 4.4.

The implementation of the Fracture Control Plan required in Clause 11 — 5 will help to ensure that steel bridges with critical tension components will serve a useful and serviceable life thoughout their design life. The Fracture Control Plan must not be used indiscriminately by designers to circumvent good engineering practice.

Fracture critical members are defined as those tension components of a bridge whose failure would be expected to result in the collapse of the bridge. The identification of such components must, of necessity, be the responsibility of the bridge designer, since virtually all bridges are inherently complex and the categorization of every bridge and every bridge member is impossible. However, to fall within the fracture critical category, the component must be in tension. Further, a fracture critical member may be either a complete bridge member, or a part of a bridge member.

Examples of complete bridge members that are fracture critical are tension ties in arch bridges and tension chords in truss bridges, provided that a failure of the tie or chord would cause the bridge to collapse. Some bridges do not depend on any single member, be it in tension or in compression, for structural integrity. Critical tension components of members usually occur in flexural members. At any given location, one flange of the flexural member must be in tension and is, therefore, a critical component if a failure of the specific flexural member would cause collapse of the bridge. The web of a flexural member is always partially in tension and similarly can be a critical component.

Members or member components whose failure would not cause the bridge to be unserviceable are not considered fracture critical. Compression members and member components in compression may in themselves be critical but do not come under the provisions of this plan. Compression components do not fail by crack formation and extension but rather by yielding or buckling. The plan provides for additional quality of material and provides for increased care in the fabrication and use of the materials to lessen the probability of fracture of tension components from crack formation and extension under static and fatigue loading.

The notch-toughness requirements for steels in bridges are similar to those used in steel bridges as specified by AASHTO [39]. The requirements developed by AASHTO were adopted after considerable research and deliberation between representatives of the AASHTO Subcommittees on Bridges and on Materials, the Federal Highway Administration, the American Iron and Steel Institute, the American Institute of Steel Construction, and various consultants. These requirements were based on numerous technical considerations that include the following [40]:

1/ an understanding of the effects of constraint and temperature on the fracture-toughness behaviour of steels that were established by testing fracture-mechanics specimens;

2/ an understanding of the effects of rate of structural loading on the fracture-toughness behaviour of structural steels;

3/ the development of a correlation between impact fracture-toughness values (K_{Id} obtained by testing fracture-toughness type specimens under impact loading) and impact-energy absorption for Charpy V-notch (CVN) impact specimens;

4/ specification of CVN impact-toughness values that ensure elastic-plastic initiation behaviour for fracture of fatigue-cracked specimens subjected to minimum operating temperatures and maximum in-service rates of loading;

5/ a verification of the selected toughness values by the testing of fabricated bridge girders that were subjected to the maximum design fatigue life, followed by testing at the minimum operating temperature and the maximum in-service rate of loading; and

6/ an awareness of the extensive satisfactory service experience with steels in bridges and an understanding of the factors that have occasionally led to brittle fractures in bridges.

The safety and reliability of steel bridges are governed by material properties, design, fabrication, inspection, erection, and usage. Both the AASHTO and OHBDC Fracture Control Plans recognize that attention to all of these factors is essential and that excessive attention to any single item will not necessarily overcome the effects of a deficiency in any other item.

Neither the AASHTO nor the OHBDC fracture toughness requirements are sufficient to prevent brittle fracture propagation under certain possible combinations of poor design, fabrication, or loading conditions. To accomplish that would require intermediate or upper-shelf dynamic toughness levels. These levels of fracture-toughness are not needed to ensure the development of the fatigue strength of the member and the resulting safety and reliability of steel bridges.

The general difference in initiation and propagation behaviour as related to fracture-toughness test results is shown schematically in Figure C11 — 5.1. The curve labelled "static" refers to the fracture-toughness obtained in a K_{Ic} test under conditions of slow loading. (The curve for intermediate-loading rate tests, which are extremely complex to run, would be shifted slightly to the right of the static curve. The AASHTO material toughness requirements were developed using an intermediate-loading rate found applicable to actual bridge structures.) The impact curve is from a K_{Id} or other dynamic test under conditions of impact loading. The difference between these two is the temperature shift, which is a function of yield strength for structural steels. In an actual structure loaded at temperature A, initiation may be static and propagation dynamic. However, there is no apparent difference between the two, because both initiation and propagation are by cleavage. If a similar structure is loaded slowly to failure at temperature B, there will be some localized shear and a reasonable level of static fracture-toughness at the initiation of failure. However, for rate-sensitive materials, such as structural steels used in bridges, once the crack has initiated, the notch-toughness is characterized by the dynamic toughness level on the impact curve and the fracture appearance for the majority of the fracture service is cleavage. If the structure is loaded slowly to fracture initiation at temperature C, the initiation characteristics will be full-shear initiation with a high level of crack toughness K_c. However, the fracture surface of the running crack may still be predominately cleavage but with some amount of shear, as shown in the lower impact curve at temperature C in Figure C11—5.1.

The use of impact or dynamic fracture tests in fracture control would predict no difference in actual resistance to fracture between temperatures A and B and only a modest difference between B and C. In fact, however, there is a considerable increase in resistance to fracture initiation between A and B and between B and C, as indicated by slow-loading tests such as K_{Ic} or COD tests. However, there is essentially no difference in the resistance to fracture propagation (i.e., crack arrest behaviour) between A and B, and the difference between B and C is modest. Thus, to prevent brittle fracture propagation in a structure by using material toughness alone (i.e., without proper control of design, fabrication, inspection, and usage), the impact toughness must be increased.

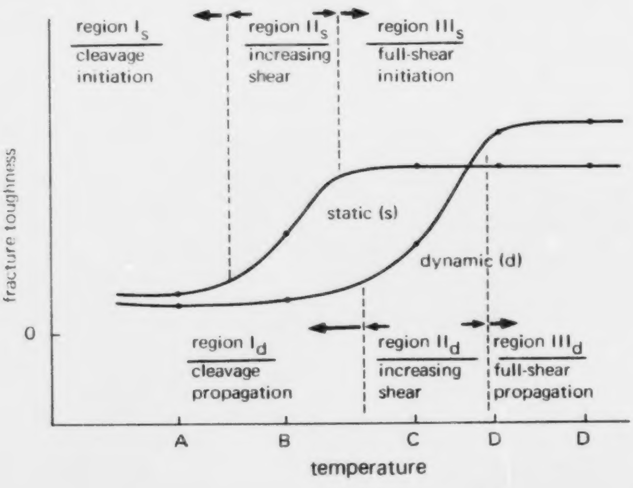
In summary, the OHBDC material toughness requirements will ensure a high level of elastic-plastic or plastic initiation behaviour for steels with fatigue cracks loaded to maximum in-service rates of loading at the minimum service temperature. Because the OHBDC Fracture Control Plan addresses all aspects that may lead to brittle fractures of fatigue failure (i.e., material properties, design, fabrication, inspection, erection, and usage), these material toughness requirements are quite satisfactory.

Composite Steel Beams, Girders, and Box Girders C11—6

Range of Interface Shear C11—6.2

Research on composite steel-concrete members in the 1960s [15, 28, 39] led to the development of the AASHTO design provision for shear connectors. Horizontal or interface shear stress range was observed to define the fatigue performance of both

Figure **Schematic Showing Relation Between Static and Impact Fracture Toughness** **C11—5.1**



stud and channel shear connectors. These design provisions marked the first time that the stress range concept was incorporated in a specification. In the AASHTO Specification, the shear connector fatigue design provisions were included in the composite design clauses rather than in the fatigue design section.

An examination of allowable interface shear stress range for welded studs and channel connectors indicated that they did not significantly differ from those permitted on fillet welds. This can be seen visually in Figure C11 — 4.3. Hence, only a single shear stress range category was used.

Additional Connectors **C11 — 6.3**

It has been the practice in North America to utilize girders having both composite and non-composite sections in continuous bridges. This permits higher allowable stress ranges to be used in the negative moment region where the presence of shear connectors would otherwise reduce the fatigue strength of the tension flange to Category c. Research on such girders has demonstrated that when shear connectors are not used in the negative moment region, additional anchorage shear connectors are required near points of contraflexure in order to prevent premature fatigue failure of shear connectors [5].

Longitudinal Reinforcement **C11 — 6.4**

When the longitudinal reinforcement develops cyclic stresses as a result of interaction with the composite section, it is necessary to limit the stress range in the negative moment regions where cyclic tensile stress range is developed. This will prevent premature failure of the reinforcement which is contributing to the composite section.

Secondary Stresses C11 — 7

Restraint at End Connections **C11 — 7.1**

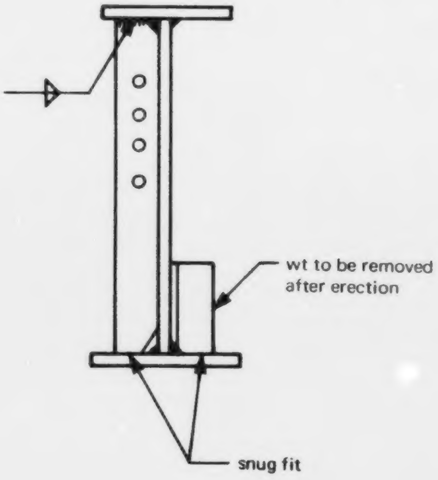
Many simple framing connections are assumed to be completely flexible. However, in reality some restraint is developed. To prevent excessive cyclic stresses in the connection angles, a modification to Wilson's Rule [32] (1940) is used so as to minimize the possibility of cracking [11].

Stress Induced by Out-Of-Plane Displacement **C11 — 7.2**

One of the major fatigue problems that has become apparent in recent years is cracking from secondary and displacement-induced stresses not accounted for in the design. The problem has developed because many bridges are essentially linear structures, and no great benefits for economy can be achieved by taking space frame action into account in the design. Although interaction may not increase economy it is a paramount importance to consider the distortions that result. A lengthy discussion of this problem and recommendations on how to avoid or minimize the problem are given in Reference 11.

In many bridges, out-of-plane movement has occurred in web gaps where floorbeam, cross frame and diaphragm connection plates are attached to longitudinal girders of skewed, curved and right bridges. The transverse connection plates

Figure **Suggested Detail for Transportation of Girders** **C11 — 7**



attached to welded built-up members should be welded or bolted to the tension and compression flanges in addition to the girder webs. This will prevent the out-of-plane movement that develops in the web gap and prevent fatigue cracks from forming as a result of that distortion.

Fatigue problems can develop in web gaps at the end of stiffeners or other restraint points as a result of handling and shipping [11]. These problems can be particularly acute when girders are shipped over considerable distances. The designer should specify or review the constructor's shipping proposals in order to minimize fatigue cracks prior to the girders being placed in service.

Figure C11—7 shows a suggested detail for use when transporting girders, which provides good support and minimizes displacements at the supports.

Dynamic Response and Restraint C11—7.3

Slender lateral bracing members are often susceptible to vibration or dynamic response and can undergo numerous stress cycles as a result of a single truck passage. This can result in cracking if points of restraint concentrate the stresses that occur. The amplitude and frequency of stress can be limited in lateral bracing by controlling the natural frequency of the member. Low natural frequency systems are sometimes susceptible to excitation by the dynamic response of the structure. A stiffer system will usually prevent such undesirable oscillations. Intermediate supports such as hangers can also be used to control the amplitude and minimize vibration. However, consideration must also be given to the end connections in order to ensure that high stresses do not result in short gap regions.

Secondary Components C11—8

Lateral Bracing, Diaphragms, and Cross Frames C11—8.2

Cross frames and diaphragms are subjected to the same frequency of stress cycles as the main members which they connect. Although such members are secondary, their contribution to the initiation of fatigue failures can be as important as that of main girder details. Careful consideration must be given to the large cyclic stresses that can result from deformation of the slab, the Vierendeel truss action of the cross-section, bracing system interaction, and the shear developed as a result of differential deformation between adjacent members.

The lateral bracing system may interact with the main load-carrying members under load. In such circumstances it undergoes a significant stress cycle during each passage of a vehicle. The resulting cyclic stresses due to live load must be considered when designing the connections in order to ensure that a premature fatigue failure will not occur.

As noted in Commentary Clause C11—7 fatigue cracks have developed unexpectedly in girder webs and in diaphragms and cross frames where out-of-plane movement has produced very high stresses in short web gaps. Examples of these cracks are given in Reference 11, which also provides recommendations on ways to avoid the problem.

Dynamic Response C11—8.3

Serious oscillations can develop in slender structural members such as hangers due to the vortex shedding effects of wind. This phenomenon is dealt with in Section 2 and the commentary thereto.

When components such as lateral bracing are susceptible to dynamic response from live load, large numbers of stress cycles can occur during the passage of a single vehicle. Hence, such members are required to be designed for over 2 000 000 stress cycles in order to prevent premature failure.

Aluminum Components — Allowable Stress Range in Fatigue C11—9

Test data on aluminum members and details are not as extensive as the experimental data base on steel connections. An extensive summary is provided in Reference 24. Additional test data have been reported more recently in References 17, 18, 21, 25 and 30. Most of these are for a zero-to-tension stress cycle and hence provide a reasonable estimate of the stress range/cycle-life relationship. Reference 17 provides a statistical analysis of several details and gives the 95% confidence limits for 95% survival. These data and the data base at Iowa State University [24] were used to estimate reasonable lower bound allowable stress range values for aluminum members and connections.

Figure C11—9 shows the stress range/cycle-life relationships that were used to define the basic stress categories given in Table 11—9.1. The details were grouped into the same categories used for steel members and connections. Hence, Table 11—4.2 provides the joint classification for both steel and aluminum members.

The design values for Category w correspond to the lower bound resistance of aluminum fillet welds in shear. The test scatter is similar to that observed in steel connections. The design values are similar to those given in the *British Code of Practice* CP 118.1969 for Class 9 joints [3]. The test data given in Reference 24 confirm the low fatigue strength of such details and proposed new British rules are comparable [18].

Reinforcing Steel **C11 — 10**

Stress Range in Reinforcement **C11 — 10.1**

Fatigue design provisions for deformed reinforcing bars were developed from NCHRP Project 4-7 and 4-7(1), and have been reported in Reference 14. This study included a literature review, an extensive experimental program, and a statistical analysis of the test results. Stress range was found to be the predominant factor influencing fatigue strength in the finite life region. A lower bound fatigue stress which also considered minimum stress was recommended and adopted by AASHTO in 1976.

The final report of NCHRP Project 4-7(1) [14] recommended that the following relationship be used to determine the permissible range of stress in the reinforcement:

$$F_{sr} = 145 - 0.33\,F_{min} + 55(r/h)$$

where: F_{min} = algebraic minimum stress level; tension positive, compression negative

r/h = ratio of base radius to height of rolled-on transverse deformation; when actual value is not known, use 0.3.

The expression given in Clause 11 — 10.1 was adopted in the interests of simplifying the fatigue design of the reinforcement, since r/h would probably not be known and an accurate value of F_{min} would often be difficult to determine and would change the stress range only slightly. This expression provides a lower bound stress range comparable to the value used prior to the adoption of the more accurate estimate of fatigue resistance.

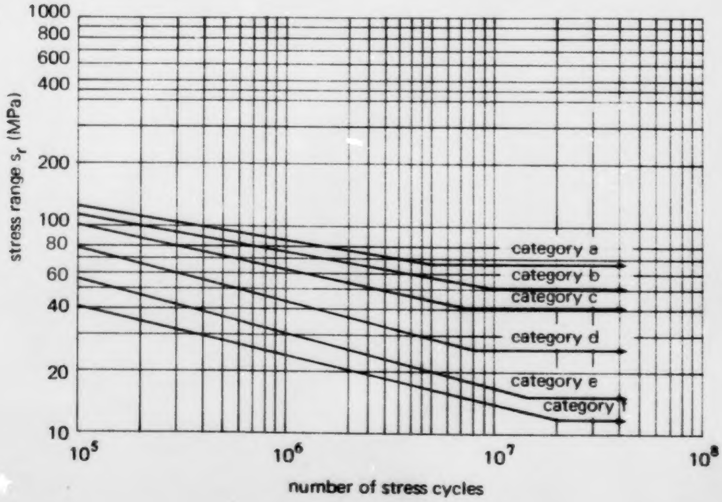
Welded Reinforcement **C11 — 10.3**

Bends and welded joints were both observed to reduce greatly the fatigue strength in the finite and long-life regions. The suggested 50% reduction in permissible fatigue stress range is in reasonable agreement with the reductions observed in limited experimental data. Unfortunately, no well-defined fatigue limit was obtained; this necessitates a conservative estimate. If possible, bends in primary reinforcement should be avoided in regions of high stress range.

Concrete Deck Slabs **C11 — 10.4**

When the slab reinforcement is also part of a longitudinal or transverse beam, the fatigue provisions of Clause 11 — 10.1 must be considered. The response of a composite girder could subject slab reinforcement to a significant cyclic stress range and the waiver provided by Clause 11 — 10.4 is not applicable.

Figure **Design Stress Range Curves for Aluminum Components and Connections** **C11 — 9**



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Appendix CA11

Welding Requirements

CA11 — 1

The welding requirements of CSA Standard W59 are applicable and are supplemented by requiring the toughness of the deposited weld metal to be greater than the toughness of the base metal which it connects. This recognizes the possibility of discontinuities in the form of porosity or slag inclusions as permitted by CSA Standard W59; it also recognizes that the weld metal may have a higher strength than the base metal.

Welding Procedures

CA11 — 2

When CSA W48 or AWS A5 specifications are not adequate to cover Charpy V-notch impact energy requirements, special tests are incorporated into the procedure qualifications. For quenched and tempered steels, certified consumables are not likely to be available so that the restrictions imposed will require careful attention to qualification and workmanship.

Repair welding for base metal is subject to careful control regardless of who is doing the repair — the producing mill or the fabricator. The requirements are analogous to procedures used for steel plate for pressure vessels.

Repair welding consists of deposition of additional weld metal to correct a surface condition, such as insufficient throat, or undercut, or procedures which required removal of weld or base metal preparatory to correcting defects in materials or workmanship. The latter procedures are divided into noncritical and critical repairs, as determined by type and size of defect.

Because virtually all weld repairs are made under conditions of high restraint, the minimum preheat/interpass temperature requirements are higher than specified for the original welding. In addition, the minimum preheat for the repair area must be continued after completion of the repair until a post-weld heating 200°C to 260°C has been completed. The post-weld heating is to enhance diffusion of any hydrogen that might be present.

Again, because of the possible high restraint situation in repair welding, a longer time interval is required between completion of the weld repair and final nondestructive testing than in the original welding.

Documentation of both noncritical and critical repair welding is required. This is to enable these areas to be given special attention when inspections are made after the bridge is in service.

Nondestructive Testing of Fracture Critical Members

CA11 — 3

For fracture critical members (FCMs) the plan recognizes that control of quality can only be the responsibility of the fabricator/erector. However, it is the prerogative of the engineer to assure himself that the quality of the product is as specified. This latter includes *quality assurance* (QA), witnessing of *quality control* (QC), *testing review* of the fabricator's documentation of his visual and nondestructive testing and duplicating any such work as is deemed necessary.

The effectiveness of radiographic testing and ultrasonic testing is determined by the size, shape, and orientation of a discontinuity. The Fracture Control Plan requires that both methods of testing be used in determining the quality of all tension butt welds. When the configuration of the material utilizing tension groove welds is not in the same plane, only ultrasonic testing is required.

The penetrating power and intensity of X-rays can be controlled by the user, but these same factors cannot be controlled by the user of gamma rays. The penetrating power of cobalt 60 (1.2 and 1.3 MEV) is so much higher than required for material thicknesses normally used in bridge construction that it is difficult to discern the small changes in thickness due to discontinuities. Iridium 192, however, has a lower and broader equivalent voltage (0.2 to 0.6 MEV) and more closely approaches the operating characteristics of X-rays. Therefore, the use of cobalt is restricted to material over 75 mm in thickness and is permitted because available sources for the thicker material are limited.

C12 Soil–Steel Structures

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Scope **C12—1**

The commonly-used conduit shapes, together with typical examples of existing soil-steel structures in Ontario are shown in Table C12—1. Other shapes such as vertical ellipses, underpasses, and high and low profile arches are also used.

Unlike other sections of the Code, this section specifies the minimum standards of construction. This is because the performance of a soil-steel structure depends as much on methods of construction as it does on design. Good construction practices for these structures are now established and it is the intent of this Commentary to emphasize the geotechnical requirements and recommended construction practice.

Definitions **C12—2**

Some of the terminology pertinent to soil-steel structures is shown in Figure C12—2.

Figure **Terminology Pertinent to Soil-Steel Structures** **C12—2**

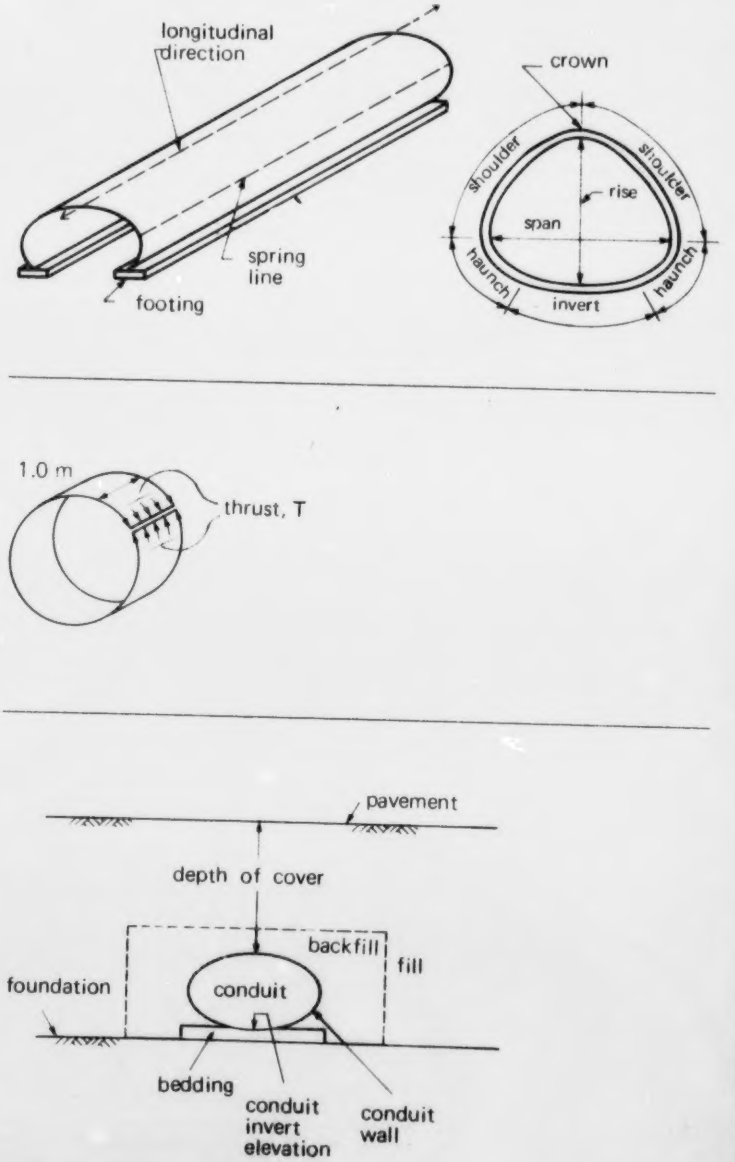
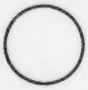
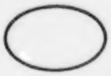
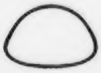
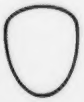
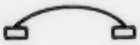
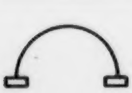
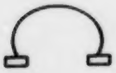

longitudinal direction
crown
shoulder
shoulder
rise
span
spring line
haunch
haunch
invert
footing
1.0 m
thrust, T
pavement
depth of cover
backfill
fill
foundation
conduit
bedding
conduit invert elevation
conduit wall

Table **Examples of Existing Soil-Steel Structures** **C12—1**

Details of Existing Installations

Cross-Section	Common Name	No.	Span, m	Rise, m	Depth of Cover, m	Metal Thickness mm	Special Features	Location	Reference or Owner	Installation Date
	Round Pipe	1	7.62	7.62	1.4	4.7		Highway 21	MTC (Thamesville)	1973
		2	6.10	6.10	7.6	4.2		Weston and Teston Rds.	Reg. Mun. of York	1977
		3	6.10	6.10		5.5		Hwy. 129 N. of Hwy. 554	MTC, Ontario	1977
		4	6.10	6.10	4.6	5.5	Twin Conduit	Gailmere Bridge	Lambton County	1966
	Horizontally Elliptical Pipe	1	8.76	4.95	1.2	5.5	Kingsize K-D**	McIntyre River	City of Thunder Bay	1973
		2	10.24	6.25	2.8	4.7 3.4	Superspan*	Port Talbot Bridge	Elgin County	1972
		3	8.89	4.37	0.8	4.7 3.4	Superspan*	Lebor Creek	County of Essex	1973
		4	7.82	4.37	1.4	5.5 4.7		County Rd. 21 (Elora)	County of Wellington	1974
		5	6.32	4.11	Varies 0.9 min	4.2	Twin Superspan	Huntingwood Drive	Borough of Scarborough	1968
	Pipe Arch	1	8.99	5.28	3.7	5.5		Bourget West Bridge	Prescott and Russell Cty.	1966
		2	7.32	4.70		4.2		LaFontaine Creek Bridge	MTC, Ontario	1967
		3	6.25	3.91	0.9 min.	4.7	conc. invert	Emery Creek	Borough of North York	1975
		4	8.31	5.61	7.9	7.0		18 Mile Creek, QEW	MTC, Ontario	1969
	'Pear' Shaped Underpass	1	8.84	8.53	1.2	6.2	Superspan*	Haley St., Renfrew City	MTC, Ontario	1973
	Part Arch	1	7.32	3.27	1.2	4.7		Thedford	County of Lambton	1963
		2	6.71	2.13		5.5		City of Sudbury	1966	
		3	6.40	2.54		2.8			Twp. of Pittsburgh	
		4	6.40	2.29		4.7		1st Concession, Perth County	Sebringville	1971
		5	6.10	2.08		6.2		Progress Rd.	Borough of Scarborough	1962
	Half Circular Arch	1	7.62	3.81	2.1	6.2		St. Paul Street Bridge	City of Brockville	1976
		2	7.32	3.66		5.5		Spruce Ave.	City of Toronto	1967
		3	6.10	3.05		3.4	Triple Span	2nd Line West of Derry Road	Region of Peel	1969
	Re-entrant Arch	1	15.29	5.49	1.5	6.2	Superspan*	Black Creek	Norfolk County	1971
		2	12.19	6.38	1.5	5.5	Superspan*	Brown Creek	Elgin County	1967
		3	11.50	5.53	2.0	6.2 5.5	Superspan*	Stouffville Pit	Consolidated Sand and Gravel	1974
		4	7.54	2.90	0.6	4.2	Twin Structures	Mead Creek, Ashburnham Drive	City of Peterborough	1975
		5	7.25	3.68	1.2	4.2		N. Veuve River, Conc. 5	Township of Hagar	1977

*incorporating patented longitudinal beams, and sometimes transverse ribs.

**incorporating a 305 mm thick relieving slab; cross-section has three radii.

Notation **C12 — 3**

A few terms are illustrated in Figure C12 — 3.

Hydraulic Design **C12 — 4**

Although the design of soil-steel structures is based primarily on the structural and geotechnical considerations specified in this section, failures of such structures, which convey water, are usually caused by washouts and/or hydraulic uplift. Experience indicates that most hydraulic failures could have been prevented by provision of either or both of the following:

a) an adequate waterway to carry the design flood;
b) adequate inlet and outlet treatment to prevent scour, erosion, undermining, percolation and piping, and/or hydraulic uplift.

Structural Design **C12 — 5**

Limit States **C12 — 5.1**

Present design practice accounts for the load-carrying capacity of soil-steel structures on the basis of their buckling load and seam strength. This concept is not changed in this section, where the limit states considered are the buckling of conduit walls and the failure of seams. Snap-through buckling is a special form of buckling of the upper plates. Therefore, the buckling formulae in Clause 12 — 5.5.2 have been developed to include considerations for snap-through buckling, particularly of the top portions of the conduit wall.

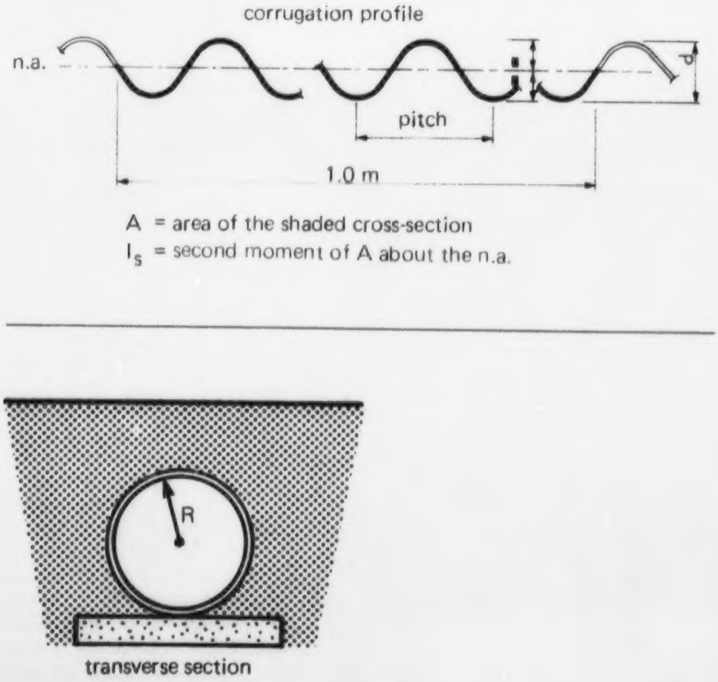
Performance Factors **C12 — 5.2**

Statistical data on the strength of the various components of soil-steel structures is, at present, insufficient for realistic estimation of the values of the performance factors. Subjective estimates of the performance factors have been made in such a way that the margins of safety of structures designed in accordance with the provisions of this section are consistent with those assumed in the design of existing successful structures.

Geotechnical Considerations **C12 — 5.3**

The design of soil-steel structures requires consideration of several geotechnical engineering aspects, such as the ability of the foundation to support the embankment fills, the long-term settlement of the foundation beneath the structure, the interaction between the backfill and the conduit wall, and soil arching due to deformations and settlement. Thus, geotechnical considerations constitute an integral, and often vital, component of the soil-steel structure design process.

Figure **Illustration of A, I_s, and R** **C12 — 3**


corrugation profile
n.a.
d
pitch
1.0 m
A = area of the shaded cross-section
I_s = second moment of A about the n.a.
R
transverse section

C12—5.3.1 Foundation Investigation

Foundation investigation should be carried out at those sites where no previous reliable knowledge exists as to the nature and extent of subsoil, groundwater, and bedrock conditions. The extent of such investigations depends on the nature and economic worth of the project, the availability of reliable ground data from geological or other sources, and the nature of the ground and foundation materials.

Subsoil conditions should be known to an adequate depth to ensure that any soft or deleterious soil layers are identified and that their potential influence on the future performance of the structure is assessed at the design stage. Should bedrock or other relatively incompressible material be encountered at a shallow depth, the foundation investigation need only confirm the continuity and reliability of such incompressible strata. The foundation investigation report should include logs of all borings and test pits made at the site, the results of laboratory tests on soil and rock samples, and a written description of the site conditions. In addition, a professional assessment should be made as to the nature, extent, and potential behaviour of the foundation materials under the proposed soil-steel structure during and after construction. Such an assessment should comment upon the stability of fills, the total and differential long-term settlements beneath and adjacent to the structure, and the time rate of such settlements. Recommendations should be made with respect to excavation of unwanted materials, dewatering, backfilling, and any in-place treatment of the foundation soils to improve their bearing and compressibility characteristics.

C12—5.3.2 Selection of Materials

"Engineered soils" refers to those materials which are used for bedding and backfilling. Such materials are selected and their use controlled by specifications relevant to the design calculations. On the other hand, "natural soils" are naturally occurring materials beneath or adjacent to the engineered soil envelope surrounding the conduit.

Since some of the design techniques recommended in this section are based upon time-independent properties of the engineered soil, it is desirable to utilize materials whose properties are indeed time-independent. For this reason, granular materials such as sands, sands and gravels, and processed or crushed granular materials are recommended for the engineered soil envelope. Non-granular or cohesive soils such as very fine sands and silts, silty clays, and clays exhibit a marked degree of time dependancy, particularly with respect to shear strength and compressibility. They are, therefore, not suitable for use as engineered soils. This section permits the use of slightly cohesive types of soil provided that their engineering characteristics (shear strength, compressibility, time-dependency) are predetermined or known at the design stage. It is implied that during construction the characteristics assumed in the design for such materials will be realized in the field; otherwise, the design has to be modified. If clayey or shaley materials are to be used, their potential long-term reaction to groundwater, freeze-thaw, and consolidation should be considered and taken into account during design. In order to reduce time-dependent effects, a non-cohesive soil should be used to replace excavated organic soils and for backfilling below the prevailing water table.

Since economical designs minimize waste or borrow, it may be necessary to utilize materials at the site which do not satisfy the criteria for engineered soils. Such materials may be incorporated in the design provided that they are utilized beyond the engineered half-span-length soil envelope zone, and provided that their use does not adversely affect the intended service function of the completed soil-steel structure. For example, marginal or waste materials may be used within approach fills outside a 1:1 line extending outwards and downwards from the pavement shoulder edges as illustrated in Figure C12—5.3.2. The outer finished slopes should be designed to ensure stable behaviour of such materials against surficial slippage or sliding. Any embankment design and frost protection criteria should, of course, be accommodated prior to the utilization of such materials.

C12—5.3.3 Soil Properties

Some soil property values can be assumed with confidence, such as the relative density (specific gravity) of solid soil constituents being taken as 2.7, without introducing large errors in the design. However, some values, such as unit weights of overburden materials, greatly influence the structural design; hence, considerable judgement is required in using assumed soil values in design. Some soil properties, such as grain size distribution and plastic and liquid limits, are needed to classify the soil properly and to facilitate material selection and use. Other soil properties are needed to model the soil in order to utilize one of the many approved design methods given in this section. Advance knowledge of the soil properties for the design procedure which is to be adopted can help considerably in conducting an effective geotechnical program.

Table A12 — 1(a) has been devised to help classify soils into three basic groups:

- Group I soils are coarse-grained and exhibit time-independent behaviour. Their moduli values are affected mainly by the degree of compaction.
- Group II soils are finer grained than Group I soils. They are basically non-cohesive in character but include soil types, such as clayey sands and clayey gravels, which exhibit a slightly cohesive matrix.
- Group III soils are fine-grained cohesive soils of low plasticity.

The values of E' given in Table A12 — 1(b) have been selected as representative of the soil type. Actual testing or field observations may be reveal E' values different from those shown.

C12 — 5.3.4 Camber

Due to its trapezoidal cross-section, a road embankment subjects the foundation beneath its centre to greater stresses than beneath the adjacent toes of the side slopes. These differential stresses in the foundation cause differential consolidation and/or "elastic" compressions and, consequently, non-uniform settlement along the length of the conduit design. This non-uniform settlement can be corrected by cambering the bedding profile in the longitudinal direction to ensure the achievement and maintenance of the designed invert gradient in the long term. In general practice, the upstream half of the conduit length is placed on an almost horizontal bedding grade, whereas the other half is placed on a sloping grade. Upon completion of settlement, the invert should assume a continuous (i.e., unbroken) profile along the conduit length.

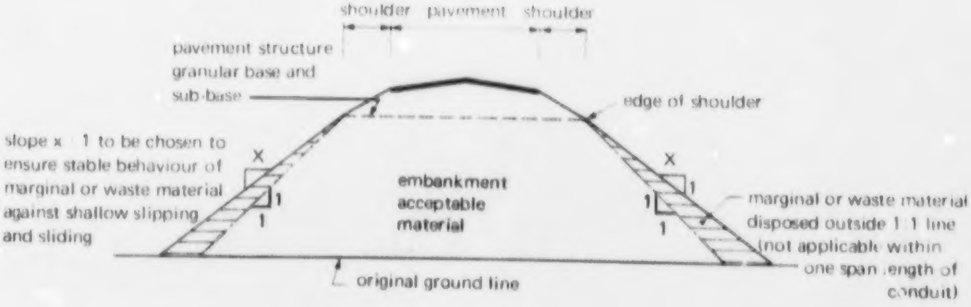
C12 — 5.3.5 Radius of Curvature

For design purposes, the interactive soil pressure at any location on the conduit is determined from the radius of curvature of the wall at a point under consideration and the thrust in the wall at that point. The ring compression theory of White and Layer (1960) [21] indicates that the wall thrust remains substantially constant over the entire cross-section and, hence, the soil pressure increases as the radius of curvature of the wall decreases. The limitation on the minimum radius of curvature in wall segments is intended to limit the maximum interface soil pressure to an amount that most soils can safely sustain when compacted to 85% or higher standard Proctor density.

C12 — 5.3.6 Arch Footings

In the design of arch footings, the passive resistance of any soil located adjacent to the footing within the zone of frost penetration should be neglected, unless the soil is granular and well drained. To accommodate wall thrust adequately, the footing may either be widened at the base or deepened, so as to increase the passive resistance

Figure **Utilization of Marginal or Waste Materials in Approach Fills** **C12 — 5.3.2**


shoulder
pavement
shoulder
pavement structure granular base and sub-base
edge of shoulder
slope x : 1 to be chosen to ensure stable behaviour of marginal or waste material against shallow slipping and sliding
X
1
1
embankment acceptable material
X
1
1
1
marginal or waste material disposed outside 1 : 1 line (not applicable within one span length of conduit)
original ground line

component. Pile footings are seldom necessary for soil-steel structures. If such footings are contemplated, the potential increase in wall thrust due to negative arching should be recognized. Negative arching could occur due to the relatively small settlement of the pile-supported structure as compared with the larger settlement of the adjoining approach fills.

Design Table **C12—5.4**

The design table provided in Appendix A12—2 may be used for selecting the thickness of conduit walls for various depths of cover, provided that the foundation for the structure is relatively unyielding, and provided that the backfill consists of a granular soil compacted to a minimum of 85% Proctor density. The table is applicable only to round pipes having spans up to 8 m. The highway live loading is applied to the embankment above the conduit. Use of this table automatically complies with Clauses 12—5.5.1.1, 12—5.5.1.2, 12—5.5.1.3, 12—5.5.2 and 12—5.5.5 but does not remove the necessity for compliance with any other clauses of this section.

Design Criteria **C12—5.5**

C12—5.5.1 Thrust

C12—5.5.1.1 General

The factored thrust in the conduit wall is assumed to be an algebraic sum of the thrust due to factored dead load and the thrust due to factored live load. In spite of the non-linear behaviour of soil-steel structures, it is considered sufficiently accurate for the purpose of design, to superimpose the separate load effects due to live and dead loads in this manner.

Details of the frame analysis method referred in this clause can be found in the report by Kloeppel and Glock (1970) [12] and the book by Széchy (1973) [19]. Details for the finite element method are in the report by Katona *et al* (1976) [11] and the paper by Duncan (1977) [7].

C12—5.5.1.2 Dead Loads

In a soil-steel structure, the thrust in the conduit wall due to dead loads is affected by the degree of soil arching provided by the system within the overburden. Positive arching, caused by the tendency for the column of soil above the conduit to move downwards relative to the adjoining soil masses, reduces the thrust in the wall. On the other hand, negative arching, in which the adjacent columns of soil tend to move downward with respect to the column above the conduit, causes the conduit wall to attract more thrust than that induced by the column of soil and materials directly above the conduit. Free field overburden pressure represents the case of no arching.

To achieve positive arching, the conduit, through its own flexibility and/or through the flexibility of its foundations, should permit greater downward movement of the soil column above it than of the adjoining soil masses. The relative downward movement of the adjoining soil masses can be minimized by ensuring that the backfill is well compacted and that the foundation is relatively unyielding. A relatively unyielding foundation is one in which positive or no arching is induced above the conduit.

For relatively unyielding foundations, the factors which affect arching and, consequently, the thrust in the conduit wall are:

a) the shape of the conduit;
b) the depth of cover;
c) the properties of the soil;
d) the stiffness of the conduit wall; and
e) the friction between the soil and the conduit.

Parametric studies reported by Katona *et al* (1976) [11] using a plane-strain finite elements program CANDE, which had been independently validated by field observed results reported by Alizedah (1977) [2] have shown that, within the range of practical values, the dead load thrust in the conduit wall remains insensitive to items (d) and (e). The results have also indicated that analysis using a linear elastic soil model and a non-linear elastic model due to Hardin (1970) [10] gave almost identical values of the thrust. Even large changes in the soil properties had little effect on the thrust. The conduit shapes and depths of cover have been determined to be the main factors affecting the values of the thrust. The ratio of the span (D_h) to the vertical distance between the crown and the springing lines ($D_v/2$) was found to characterize best the shapes of pipe arches and elliptical conduits.

The values of the shape factor μ_1 were developed for this section from the plane-strain finite element program CANDE. The results are applicable for the following range of elastic properties:

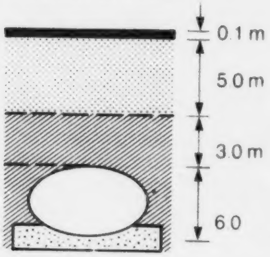
- modulus of elasticity of soil, Es = 6.2 to 9.7 MPa;
- Poisson's ratio, ν = 0.30 to 0.40.

The value of μ_1 for depths of cover less than 1 m may be obtained by extrapolation of the curves in Figure 12—5.5.1.2. Any resulting inaccuracy is likely to have minimal influence on the end result due to the large effect of vehicular live loading at such shallow depths of cover.

It should be noted that the factor μ_1 does not directly indicate whether the arching is positive or not. Positive arching is possible even when μ_1 is larger than 1.0.

The value of the dead load factor for soil, $\alpha_D = 1.25$, may be used for all materials above the crown. An example of the calculation of σ_{vo} is shown in Table C12—5.5.1.2. Note that the first expression in the clause contains the number 1000 in the denominator because R_t is in millimetres and the other length units are in metres.

Table **Example of the Calculation of Free Field Overburden Pressure** **C12—5.5.1.2**

Layer No.	Section	h_i, m	γ_{si}, kN/m³	h_i, γ_{si}, kN/m²
	Asphalt Pavement			
1		0.1	23.5	2.4
2		5.0	21.0	105.0
3		3.0	20.0	60.0
4				

$$\sigma_{vo} = \sum_{i=1}^{3} h_i \gamma_{si} = 167.4 \text{ kN/m}^2$$

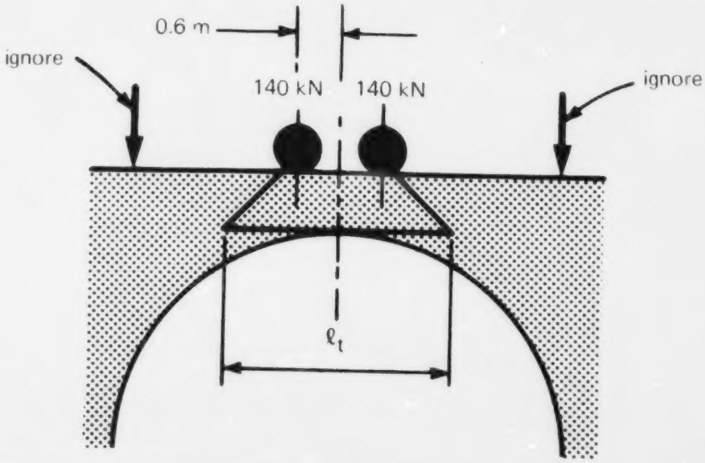
C12—5.5.1.3

Live Loads

The rationale for this clause is given in the paper by Bakht (1981) [3] which provides details of several tests on actual soil-steel structures. The design vehicle is placed longitudinally as shown in Figure C12—5.5.1.3(a). The rectangular area over which the load is assumed to be uniformly distributed is obtained as shown in Figure C12—5.5.1.3(b). The values of $\sigma_L m_f(1 + I)$ may be obtained directly from Figure C12—5.5.1.3(c).

For depth of cover H up to 1.3 m, the factored distributed load $\sigma_L m_f(1 + I)$ is maximum when there is only one vehicle on the embankment. For larger values of H, the maximum values of the factored distributed load are obtained when there are two

Figure **Positions of Vehicle for Maximum Live Load Thrust** **C12—5.5.1.3(a)**



side-by-side design vehicles on the embankment. The factored distributed loads due to three or more vehicles are usually smaller than those for two vehicles; hence, more than two design vehicles need not be considered.

C12—5.5.2 Strength Requirements

The live load and dead loads acting on a soil-steel structure are carried mainly by the steel conduit wall. This develops compressive forces and bending moments in the conduit wall. In common with the corresponding provisons of the AASHTO Specifications, the provisions of this Code are based on the assumption that for design purposes the wall may be assumed to be subjected to compressive forces only, i.e., to be in a state of ring compression. The justification of this assumption lies in the manner in which the interface pressure between the steel conduit wall and the surrounding soil mass changes with movement of the wall. If a bending moment occurs locally which is high enough to cause partial yielding of the conduit wall, the resultant movement of the conduit wall causes an increase in the interface pressure developed in the adjacent soil mass and this increase in pressure tends to inhibit further movement.

The failure of a conduit wall under thrust alone may be due to wall crushing, when the compressive stress $\sigma = T/A$ reaches the yield point; or by elastic buckling; or by a combination of the two.

For the purpose of calculating the elastic buckling load, the conduit wall is divided into the following zones:

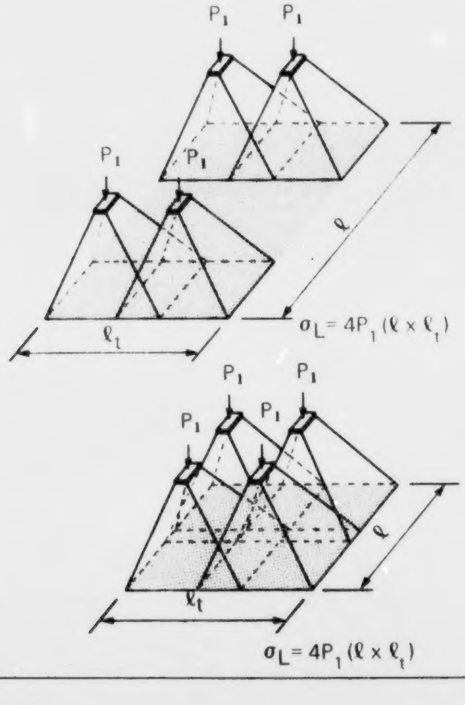
a) lower zone, in which the radial displacements of the wall are towards the soil;
b) upper zone, in which the radial displacements of the wall are towards the inside of the conduit.

These two zones are separated by points of zero radial displacement which are situated at positions defined by θ_o from the vertical, the origin being at the centre of the arc at the crown. Approximate values of θ_o are given by Abdel-Sayed (1978) [1] as:

$$\theta_o = \left(1.6 + 0.2 \log \frac{E2}{E'R^3}\right)$$

In circular and horizontal elliptical conduits with constant wall thickness throughout, the elastic buckling of the upper zone governs the design. In other cases where the conduit shape and/or wall thicknesses vary, it is necessary to check for elastic buckling of the lower zone as well.

Figure **Dispersion of Concentrated Load Through Fill** **C12 5.5.1.3(b)**



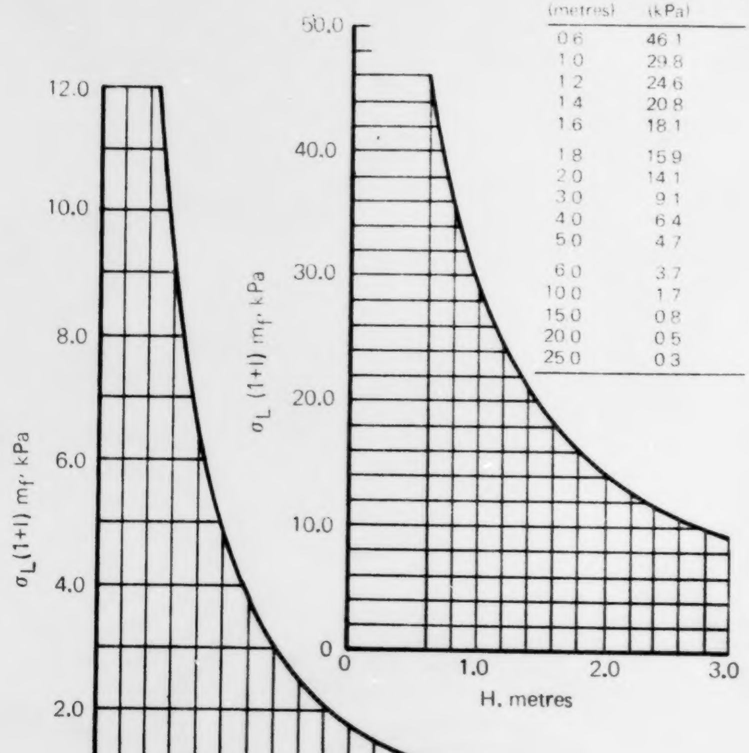
The first equation given in Clause 12—5.5.2 for f_b where $R \leqslant \bar{R}$ relates to inelastic behaviour, whilst the second relates to elastic behaviour. So far as the lower portion of the wall is concerned, these two equations are identical with those used in the AASHTO Specifications and by research workers in the field of soil-steel interaction such as Booy (1957) [5] and Meyerhof and Baikie (1963) [15]. An additional aspect of the first two equations in Clause 12—5.5.2, which has no counterpart in the AASHTO Specifications, is that they have been made applicable to the upper portion of the wall also; this has been achieved by increasing the value of λ, and hence of K, to allow for the effect of the tangential movement of the points of zero radial displacement. In the case of the lower portion of the conduit wall, where radial movements are towards the surrounding soil mass, tangential movements at the points of zero radial displacement have relatively little effect on buckling behaviour. By contrast, such movements can have a significant effect on the buckling behaviour of the upper portion of the conduit, where radial movements are away from the surrounding soil mass. It is because of this difference in the behaviour of the two portions of the conduit wall that two expressions for λ are needed in the equations.

The inelastic buckling is considered to start when f_b is equal to half the yield stress. A parabolic equation is assumed for the transition zone between full yielding ($KR/r = 0$) and the elastic zone at $f_y/2$.

Sectional properties, such as the second moment of inertia (I_s), cross-sectional area (A) and radius of gyration (r) for structural plate corrugated steel pipe (SPCSP) products may be obtained from CSPI Bulletin No. 601-79 (Revised 1972) [6].

In most cases, it will be found that for spans larger than about 8 m it may not be possible to satisfy the requirements of this clause using the 152 × 51 mm corrugation

Figure **Dispersion of Live Load Through Fill** **C12—5.5.1.3(c)**



H (metres)	$\sigma_L (1+I)\, m_f$ (kPa)
0.6	46.1
1.0	29.8
1.2	24.6
1.4	20.8
1.6	18.1
1.8	15.9
2.0	14.1
3.0	9.1
4.0	6.4
5.0	4.7
6.0	3.7
10.0	1.7
15.0	0.8
20.0	0.5
25.0	0.3

profile plate alone. For spans larger than 8 m, the addition of special features, as mentioned in Clause C12—9, may be necessary. Large span soil-steel structures without any special feature are rare.

C12—5.5.3 **Seam Strength**

The ultimate strengths of seams for various bolting arrangements shown in Figure 12—5.5.3 are based on seam strength tests conducted on uncurved short columns by the Utah Department of Highways in 1964 and by Pittsburg Testing Laboratories in 1965. Similar values are also used in the AASHTO Specifications and in the AISI *Handbook of Steel Drainage and Highway Construction Products* [9].

C12—5.5.4 **Handling and Installation Strength**

An upper limit of the flexibility factor has traditionally been used as an indication of the minimum stiffness of the conduit wall so that it can be handled and installed without inducing any permanent set. The value of 0.114 mm/N as an upper limit has been adopted from AASHTO Specifications. This provision may be set aside, however, if with improved methods of construction it becomes possible to erect a conduit wall having flexibility factors in excess of 0.114 mm/N without inducing permanent set.

Durability

C12—6

Successful long-term performance of a large number of soil-steel structures validate the specifications cited in this clause. The same specifications are current in the Ontario Ministry of Transportation and Communications.

Environments which may be aggressive to soil-steel structures are identifiable by their low pH and/or soil resistivity. Simple field test methods have been developed in California to identify the aggressiveness of a conduit environment with a view to estimating the number of years to first perforation in galvanized steel; these are reported by Beaton and Stratfull (1962) [4]. The California prediction graphs should be conservative for the Canadian climate. Due to favourable performance of culverts in Canada, little research has been done to correlate the California or other methods to the prediction of total service life in colder environments.

If service life is believed to be a design consideration in those instances where a particularly aggressive environment has been identified, or where results of prior experience in such an environment are not reliably determinable, the designer may wish to apply the California method to estimate the years to first perforation. First perforation of the steel wall does not mean loss of structural integrity. The useful life of the structure will exceed the number of years to first perforation by an extent of time dependent on the maintenance given to the structure after appearance of first perforation.

In normal service, under average environmental conditions, galvanized steel walls are expected to provide a minimum maintenance-free service life of 50 years. Generally speaking, the service life failure of zinc-coated steel occurs due to atmospheric corrosion at the air-steel-water interface. Protection of this "splash zone" with a greater thickness of steel, or coatings of other materials, will extend the design service life.

Extended service life for soil-steel structures may be designed by specifying protective coatings. A wide range of exterior coatings and interior invert pavements is available. Typical of these are:

a) bituminous coatings, the process comprising plant dipping in hot asphalt;
b) bituminous coatings, as above, in conjunction with field paving on the inside surface of the conduit wall;
c) field application of asphalt mastic, by spraying or brush or trowel application;
d) concrete paved inverts, which are usually installed after backfilling and settlement has taken place.

Special Design Considerations

C12—7

Minimum Depth of Cover

C12—7.1

The specified normal minimum depth of cover takes into account:

a) The need to keep bending moments in the conduit wall, due to live loads, down to a level at which they may be safely neglected in the design.
b) The dynamic load allowance, which is based upon the stated depth of cover. The effect of any decrease in depth of cover on this allowance is uncertain.
c) The danger of upheaval of a soil wedge above and to one side of the conduit due to the application of a large surface load on the other side of, and eccentric to the conduit in a transverse plane.

Multiple Structures

C12—7.2

There are two principal reasons for requiring the specified minimum clearance between adjacent conduits. The first is the need to introduce compacting equipment between the conduits to permit adequate compaction. The second is the need to avoid a situation in which a soil mass contained between the inside shoulders of the two conduits could fail by upheaval as a consequence of two surface loads being applied simultaneously which are eccentric to, and are on either side of the two conduits.

Cut Ends **C12 — 7.3**

Cut ends may be required for a variety of reasons, including aesthetics. The provision of cut ends involves a local reduction in structural strength because of loss of continuity of the closed cross-section; hence, it is necessary to make careful requirements in regard to them. The recommendations of the CSPI *Technical Manual* (McCavour [14]) constitute preferred practice for installation of such cut ends.

Structures in Earthquake Zones **C12 — 7.4**

When soil-steel structures are to be constructed in earthquake-prone areas, the design must take into consideration two potential problems, namely:

a) instability of cuts within, and embankments or fills founded upon sensitive cohesive soils;

b) liquefaction of non-cohesive soils.

For most practical applications, the use of a horizontal ground surface acceleration equivalent to 0.1 g will ensure that the stability analysis results in an adequate slope design for fills and cuts constructed upon sensitive cohesive soils such as Leda Clays. The analytical method is pseudostatic in nature and its limitations have been documented by Seed (1967) [17]. More refined methods do not appear justified for use, at this time, since reliability considerations of seismic data and subsurface conditions outweigh any benefits of more accurate analyses.

Liquefaction is a phenomenon in which a cohesionless soil loses strength during an earthquake and acquires a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet. This phenomenon is reported, for example, by Seed and Idriss (1971) [18]. A review of soil liquefaction procedures, presented by Valera and Donovan (1977) [20], may be consulted to assess liquefaction potential at a given site.

Construction and Installation C12 — 8

General **C12 — 8.1**

Construction and installation procedures have a profound influence on the success of soil-steel structure design. For this reason, their specification by the engineer is made a requirement.

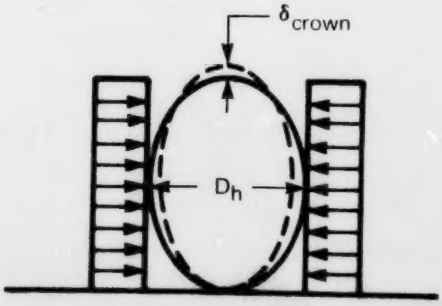
The paper by Mirza and Porter (1981) [16] should be consulted for good construction practice for soil-steel structures.

Deformation During Backfilling **C12 — 8.2**

During the initial stages of backfilling, when the conduit is not fully contained by the soil, the conduit wall undergoes high deformations. These, if allowed to exceed certain limits, will induce permanent set in the wall. The provision in this clause is aimed at restricting such excessive deformations.

The limits of vertical crown deflections for round and elliptical conduits were derived by an analysis in which the conduit walls, with a 152 mm × 51 mm corrugation profile were subjected to uniform lateral pressures as shown in Figure C12 — 8.2. Limits of the crown deflections, which could be characterized by the span-to-rise ratios of the conduit, were established so that the stresses anywhere in the cross-section remained within 90% of the yield stress of the steel. Values of the nondimensional parameter μ_2 given in Figure 12 — 8.2 were calculated from the results of the analysis.

Figure **Conduit Wall Subjected to Uniform Lateral Pressures** **C12 — 8.2**



For shapes other than round and elliptical, the limit of the vertical deflection is based mostly on practical considerations rather than on analysis.

The crown deflection limits specified in the clause are meant for construction rather than design control. If the deformations begin to exceed the specified limits, measures should be taken to contain them. With the present state of the art it is difficult to "design" a conduit wall so that it can comply with the provision of this clause.

Foundation **C12—8.3**

Foundations with reasonably uniform settlement properties are necessary to avoid situations where the invert may be founded partially on compressible materials and partially on incompressible materials. Lack of uniformity along the invert bedding could induce undesirable stress concentrations in the conduit wall above the incompressible areas. A condition in which the foundation under the conduit is less compressible than that of the adjacent areas should be avoided. In such conditions, columns of soil adjacent to the conduit settle more than the column of soil above the conduit, thus inducing negative arching which, in turn, increases the thrust.

Foundations with fairly uniform settlement properties can be provided by adjusting the bedding thickness, replacing compressible materials, shattering bedrock, and other similar treatments.

Bedding **C12—8.4**

The bedding serves to receive from the conduit and transfer to the foundation the loads applied both during and after construction. The use of either flat or shaped bedding is permitted. A shaped bedding is preferable for the larger sized pipe arches. A flat bedding renders compaction under the haunches difficult for large conduits. Preshaped bedding interferes with the assembly of field-bolted structures; therefore, proper field bolting of the invert plates becomes the criterion for shaping the bedding. If shaped bedding is selected, a uniform blanket of uncompacted bedding material of 200 mm thickness should be provided so as to allow proper embedment of the invert corrugation profile. It is important that the bedding material be in full contact with the entire bottom portion of the conduit.

The width of the bedding in open fill, in contrast with a trench condition, should be wider than the span to encourage even settlement of the structure and sidefills and improved pressure distribution around the conduit.

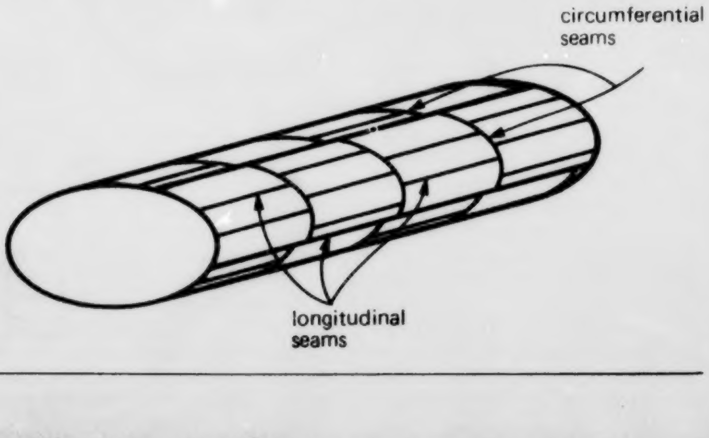
Assembly and Erection **C12—8.5**

Assembly instructions and drawings are provided by the structural plate corrugated steel pipe manufacturer. These should be made part of the plans. The conduit may be either pre-assembled and moved on to a prepared bedding or it may be assembled in a bottom-side-top plate bolting sequence.

The torque requirement for bolting of longitudinal and circumferential seams as given in Figure C12—8.5 does not refer to re-tightening of the bolts. General practice is to tighten the bolts to specification torques during assembly and erection. It is recognized that some relaxation and easing of the torque will occur after installation. The requirement of the Code is to ensure that the initial torquing has been reliably carried out. If plate ends in the seams are kept reasonably parallel during assembly and bolt torquing, closure of the conduit walls is seldom a problem. If unusually high force is required to effect closure, the designer and the manufacturers should be notified.

In order to guard against permanent set in any portion of the wall, the following sequence of construction is recommended.

Figure **Illustration of Longitudinal and Circumferential Seams** **C12—8.5**



a) The walls should be assembled using a minimum number of bolts until all the plates are in place. Normally, it should be possible to erect a segment of the conduit with as few as 3 to 6 bolts along each seam, untightened, and placed near the centre of each plate.

b) After overall "loose assembly," the remaining bolts should be inserted, working away from the centre of a seam towards the corners of the plates. Corner bolts should be inserted last, and only after all other bolts are in place and nominally tightened.

c) Final torquing should be done on one circumferential seam after another, starting at one end of the structure.

d) For long conduits, the sequence of final torquing may be provided by the fabricator to avoid the possibility of a cumulative rotation of the cross-section which might occur.

The conduit walls of some structures tend to deflect excessively during assembly, due to their own dead weight. For such structures, struts are sometimes placed inside the conduit to maintain its shape during construction. The struts, however, should not restrict crown movement. During earlier stages of backfilling when the crown begins to move upwards, the struts should allow free upward movement of the crown. In no case should struts be allowed to restrict the downward movement of the crown during the later stages of the backfilling operation. Prevention of downward crown movement inhibits the development of passive soil pressure at the sides of the conduit.

Backfill **C12—8.6**

Current practice is to limit backfill lifts to 300 mm of uncompacted thickness. The Code permits the use of thicker loose lifts by specifying 300 mm compacted thickness. This change is meant to encourage greater utilization of more efficient compaction equipment. Current practice requires that the backfill be raised evenly on both sides of the conduit. The Code has eased this requirement to some extent to encourage efficiency in construction by permitting an imbalance of two compacted lift thicknesses on either side of a conduit as long as the deformation criteria of Clause 12—8.2 are met.

The use of heavy vibratory equipment is not being allowed close to the conduit in order to limit the development of incremental earth pressures that may cause excessive deflection of the conduit walls during backfilling. Under compactive effort, earth pressures often exceed the at-rest values. During placement and compaction of backfill material, all equipment must be specified to operate parallel to the longitudinal axis, unless special circumstances or constraints of space dictate otherwise. Compaction parallel to the longitudinal axis produces less incremental earth pressure loading than compaction perpendicular to the longitudinal axis.

If the structure is to be assembled and backfilled before work on the adjoining approach is commenced, the backfill should extend along both sides at least one span distance and should be brought up to provide a cover of at least 0.6 m or one-sixth of the span, whichever is the greater. This provision is meant to ensure the retention of the conduit in its original position and shape when the adjoining embankments are subsequently built.

Headwalls and Appurtenances **C12—8.7**

Hydraulic uplift and erosive forces are known to affect the stability of inlets and outlets of soil-steel structures conveying water. Such hydraulic forces are especially significant for steep gradients and high flow velocities. Where potential headponding may occur, the possibility exists of percolation flow through the backfill along the conduit wall. Therefore, appropriate end treatments should be considered to ensure satisfactory hydraulic performance.

The end treatments may consist of:

- cut-off walls of metal, masonry, or concrete to prevent percolation, scouring, and/or uplift;
- embankment shaping to improve flow characteristics;
- slope treatment to reduce scour and erosion;
- clay seals to prevent infiltration, percolation, erosion, and scour.

Special Techniques **C12—9**

Special design and construction techniques thought to benefit improved soil-structure interaction are illustrated in Figure C12—9. It is to be noted that some of these techniques may be patented.

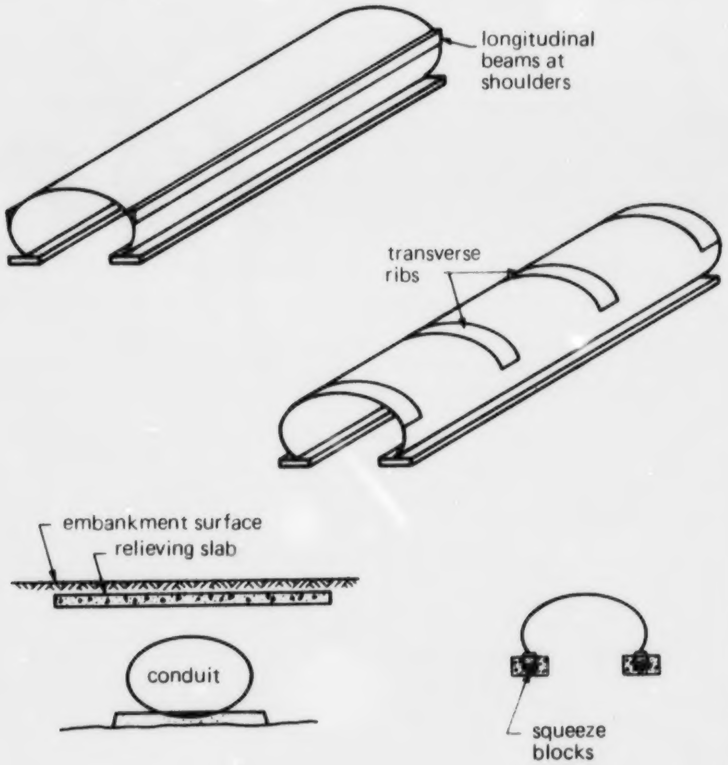
The logitudinal beams connected to the conduit wall at the soil-steel interface along the shoulder are usually constructed of reinforced concrete. Backfill is brought up on both sides of the structure to the underside of the beam placement location prior to installation of the reinforcing steel and concrete formwork.

Relieving slabs are generally cast close to the crest of the structure, often at the minimum depth of cover, as a means of obtaining a low profile gradient above the structure. A case history of an instrumented installation is reported by Fung (1974) [8].

Transverse ribs may consist of steel I-beams or sections of structural plate corrugated steel pipe. SPCSP sections are generally placed crest-to-crest along the top radius of the structure. Their width and spacing along the longitudinal axis is determined by analysis or from prior experience with structures of similar geometry.

Treated spruce timber blocks have been incorporated on a re-entrant arch installation as reported by Lefebvre et al (1976) [13]. Their purpose is reportedly to serve as "squeeze blocks" so as to induce and maintain positive arching over the long term in such arches.

Figure **Special Techniques to Improve Soil-Structure Interaction** **C12 – 9**


longitudinal beams at shoulders
transverse ribs
embankment surface
relieving slab
conduit
squeeze blocks

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C13 Wood Structures

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Scope

C13—1

This section applies to the types of wood structures currently designed and built by the Ministry and those in current use in Ontario such as glued-laminated girders, timber stringers, transversely-laminated decks, longitudinally-laminated decks designed to act compositely with concrete, prestressed longitudinally-laminated decks, and trusses. This section does not cover falsework and formwork.

Because of the limited amount of test data available, the procedures of CSA Standard 086 have been adopted for the design of fasteners. These design procedures have been adopted by reference.

For structures not covered by this section, the designer may refer to other codes for the design requirements and in such cases approval is required. The scope of the Code is limited to those conditions for which performance factors are given in Table 13—4.4.

Limit States

C13—4

General

C13—4.1

To achieve the service life required for a structure, consideration has been given in specific clauses in this section to the following.

a) **Durability of the Wood**
   Preservative treatment is specified to provide protection against decay and infestation by wood-destroying insects.

b) **Durability of Metalwork, Hardware and Fasteners**
   Hot-dipped galvanizing or other protective treatment has been specified to prevent corrosion.

Attention should be paid in detailing to the avoidance of moisture traps.

Serviceability Limit State of Deformation

C13—4.2

Under load, wood is subject to increasing deflection with time, the amount depending on the species, magnitude of stress, moisture content at the time of loading and subsequent changes in moisture content that take place under load. Slip and rotation of mechanical joints also exhibit the same characteristic. Care should be taken to avoid adverse effects arising from the deformations. A deflection limit is specified since the working of the elements and joints due to repetitive excessive deflections would lead to progressive deterioration of the elements and joints, and to probable asphalt wearing surface distress.

Performance Factor **C13 – 4.4**

Performance factors, sometimes designated as resistance factors, for wood design are chosen to represent a measure of the precision of the prediction of the corresponding limit state of the constructed work, the accuracy of the specified strength values used, the variability of the specified strength values, and the consequences of exceeding the limit state. They yield a safety index of 3.5, except that safety index values range around 3.0 for single member cases. The lower value of safety index is only significant for failure sensitive single member cases such as single leaf trusses and those beams for which failure is critical to the overall structure.

Values of unity for compression perpendicular to grain represent very small consequences of failure. The Code is not intended to cover cases where no performance factors are given.

Performance factors appear relatively high because specified strengths are based upon the 5th percentile strengths which are significantly less than mean strengths due to the large inherent variability.

General Design **C13 – 5**

Design Assumption **C13 – 5.1**

Plastic redistribution is not to be assumed in the analysis for load effect.

Spans **C13 – 5.2**

Effective span length may be larger than the clear span because of compression of the fibres at the supports in edge bearing.

Duration of Load Effect **C13 – 5.3**

In the past, the concept of duration of load has been developed from the testing of small clear wood specimens in accordance with ASTM Standards. It was projected that the long-term strength, using 50 years as a base, was 9/16 of the short-term strength of clear wood.

During the 1940s the base was moved from 50 years to 10 years duration which was reflected by a further multiplication of 11/10. The conversion from short-term loading to 10-year loads became:

$$\frac{9}{16} \times \frac{11}{10} = 0.60$$

i.e., the breaking strength of a small clear specimen loaded for 10 years is taken to be 60% of that of the same specimen loaded for five minutes. However, in-grade testing with commercial lumber indicates that full-size members behave differently from small clear specimens. Madsen and Barrett [1] showed that the strength reduction for periods of up to about two years was not as great as previously thought.

In addition, it should be noted that most loads do not act at their maximum intensity for very long periods, even when calculated as the accumulated time over the lifespan of the structure. As an example, consider a logging bridge on which a maximum load effect occurs every 5 min during the day. Assuming, conservatively, that maximum stress occurs while the vehicle is moving at 50 km/h on the bridge, and that the bridge is 30 m long, one finds that the loaded time is 2.2 s per passing. Over a 50 year life, this amounts to 30 days, assuming 250 working days per year. Similarly, snow loads and other live loads do not last at their maximum intensities for very long.

For the above reasons, in this Code the time base has been changed to one month duration, which covers almost all design conditions. Tests [2] indicate a 20% reduction in short-term strength for a two-month period is adequate. This Code is based on a 15% reduction reflecting that design load maxima generally occur less frequently.

The stresses presented in this Code are thus for one month duration, and reduction for the exceptional cases where the maximum load in fact lasts longer than one month is 30% of the stated values. This 30% reduction factor need only be applied to the load effects due to sustained loading such as earth pressure and dead load.

When combined sustained and one-month loading conditions occur, the members may be considered as having resistance based on 100% of the stated values for the combined loading.

It is the intent to use the one-month value also for cases where the loads last for shorter periods, since the reduction is 15% from short-term to one month and the adjustment for a shorter period of time would be only a fraction of the 15%.

Historically, impact in wood has been ignored because of the traditional concept of the shape of the duration of load curve for duration less than 5 min [3]. Current research indicates that no increase in strength for impact should be used [1].

Size Effect **C13 — 5.4**

In-grade testing has shown that size effects exist in wood, i.e., the capacity of a member is dependent upon the volume of wood and the level to which it is stressed. This applies to bending, tension, shear, and bearing. This section takes size into account by providing adjustment factors which are to be applied to the specified strength values. For dimension lumber and sawn timber the base has been chosen to be 286 mm deep members for flexure and tension. A volume of 1.0 m³ has been chosen as the base for shear in general and for glued-laminated components in flexure. The size effect for glued-laminated components in tension is not well established but the specified tension values on the gross section have been reduced as a measure to allow for this.

Moisture Content Effect **C13 — 5.5**

Research has shown that for flexural and tension members strength at the 5th percentile level is not affected by drying [4, 5]. In compression, a reduction of about 20%, for wet service conditions, has been incorporated in the specified strengths [6].

Load-Sharing Effect **C13 — 5.6**

This clause has been written to take account of the increased strength of a multi-member system resulting from load sharing. This is due in part to:

- the reduced probability of occurrence of low-strength material in adjacent laminations due to grouping of individual members into structures;
- increase in effective strength due to joining of members into a statically indeterminate structure;
- increase in effective strength due to increase in effective homogeneity whenever zones with defects are locally reinforced by neighbouring members. This is a local effect and concerns "in-member" variability.

The extent of the benefit resulting from this has been studied mathematically, in particular by Zahn [7].

Limited test results and mathematical predictions have been reported [8]. These test results agree, in general, with Zahn's mathematical predictions and have therefore been used as a guide in establishing the load-sharing factors in Clause 13 — 5.6.

The high load-sharing coefficient of 1.5 for prestressed wood decks was derived from in-grade testing [9].

Notched Components **C13 — 5.7**

Notches are known to be stress raisers, particularly when compounded with fluctuating loads and impact. It should be emphasized that, in addition to the stress raiser effect of the notch, most notch geometries result in tension perpendicular to grain; this should be avoided.

Flexure **C13 — 6**

Size Effect **C13 — 6.2**

C13 — 6.2.2

Glued-Laminated Beams

Data on the bending strength of glued-laminated beams indicate that the modulus of rupture decreases with increasing size. Preliminary examination of available data for size effect was presented in Reference 10. These data and some additional data from recent tests provide a basis for the empirical expression for size effect given in this clause. It has been adjusted to give a size effect value of unity for beams of volume 1.0 m³. It does not account for the fact that size effect actually varies with the shape of the moment diagram.

Lateral Stability **C13 — 6.4**

The design of flexural members must account for the possibility of lateral instability. The behaviour is affected by beam proportions and type of end restraint and compression edge restraint.

C13 — 6.4.1

This clause establishes beam proportions where the question of lateral support need not be considered.

C13 — 6.4.2

When the depth of a beam exceeds the breadth and there is no lateral support, the load-carrying capacity of the beam may be reduced. Stability formulae have been derived for a beam supported and held vertically at its end. To satisfy the analysis requirements, a beam must be held vertical at its ends as specified. Drift pins, decking, bulkhead planking, or anchor bolts may provide sufficient support.

Slenderness Factor of a Beam **C13 — 6.5**

This clause defines the slenderness factor for beams and is patterned after a similar clause in Reference 11. The unsupported length L_u of the beam is the distance between the points of effective lateral support of the compression edge of the beam or between the bearing supports.

Intermediate Beams, and **C13 — 6.7**

Slender Beams **C13 — 6.8**

In the procedure of Reference 12, the transition from intermediate to long beams occurs when the reduction factor on f_{bu} reaches 2/3. The equations which appear in the two clauses have been modified from conventional working stress procedure by:

- replacing the allowable bending stress by the specified bending stress f_{bu};
- replacing the mean modulus of elasticity E_{50} by the 5th percentile value E_{05}; and
- replacing the working stress factor by an ultimate strength factor of 0.70 taken from Reference 12.

The overall result of the above changes is that the fraction 3/5 which appears in the working stress formula for C_k has become 1.0. The definition of effective length has been changed to correspond to actual lengths for a simple beam with equal end moments.

Shear C13 — 7

Sawn Wood C13 — 7.1

The shear strength of beams with end splits may be governed by a failure mode involving critical crack growth. The application to wood beams is detailed in Reference 13. The critical shear stress at this failure mode is given in that reference as:

$$\tau_{crack} = K_{\pi c} \left(\frac{11.25}{d}\right)^{0.5} \left(\frac{3}{1 + 2\,a/d}\right)(0.13) \qquad (1)$$

with additional modifiers for duration, safety, etc. For a reference beam depth of 11.25 inches, $K_{\pi c}$ is the stress intensity factor, a is the length of end split, measured from centreline of support to tip of split, and d is the beam depth (all dimensions in inches).

The value of $K_{\pi c}$ at the 5% fractile of its probability distribution may be taken as approximately the mean τ_{ASTM} from standard shear block tests. The Commentary for Clauses 13 — 11.2 and 13 — 12.2 indicates how specified strength values for shear were derived from shear block values. $K_{\pi c}$ values are 8 times the specified strength values. Thus we may write:

$$\tau_{crack} = 8f_{vu} \left(\frac{11.25}{d}\right)^{0.5} \left(\frac{3}{1 + 2\,a/d}\right)(0.13) \qquad (2)$$

Equation 2 was developed for the case of uniformly-loaded beams. In the case of stationary concentrated loads acting on beams with end splits, the value of k_v, the size effect factor of Clauses 13 — 7.1.2.1, 13 — 7.1.2.2 and 13 — 7.2.2.2, may be derived using Figure 4 and its accompanying equation from Reference 14. There is no available research on the effects of moving loads.

The k_v values of Clause 13 — 7.1.2.1 are derived from Reference 14 for wood graded as select structural as well as for No. 1 using the NLGA grading rules [22], and assuming a beam depth of 750 mm with critical shear taken at d from the centreline of the support. The projection of the beams beyond the centreline of support is assumed to be about 150 mm. Using these assumptions and Reference 14, the following values of k_v are obtained, where d is expressed in millimetres:

$k_v = \dfrac{45}{\sqrt{d}}$ for wood graded as select structural and for No. 1 graded as select structural for end splits only.

$k_v = \dfrac{30}{\sqrt{d}}$ for wood graded as No. 1

For analysis of members with existing end splits, the above k_v values may be used to derive a general formula for k_v which may be expressed closely as follows:

$$k_v = \frac{75}{\sqrt{d}} \cdot \frac{1}{(1 + 2a/d)} \qquad (3)$$

where a and d are expressed in millimetres.

The beam nominal resistance becomes:

$$V_u = \frac{f_{vu}}{1.5}\, bd\, k_m\, k_v \qquad (4)$$

Glued-Laminated Timber C13 — 7.2

Shear in glued-laminated beams is treated in accordance with the procedure to account for size effect as outlined in References 14, 15, and 16, the last of which defines a reference shear stress τ_0 as:

$$\tau_0 = \frac{1.5\,V_0}{bd} \qquad (5)$$

where V_0 is the total load on the span. Then τ_0 at failure is given by:

$$\tau_0 = \frac{\lambda\tau'}{(bdL)^{1/\kappa}} \qquad (6)$$

where λ is a load effect parameter; τ' is the shear strength of a volume 1 cu. in.; b, d, and L are width, depth, and span length, respectively; and κ is a parameter of the probability distribution of shear strength, related to the variability by the approximate equation:

$$\text{coefficient of variation} \simeq 1.2/\kappa$$

κ is taken herein as 5.53.

Combining Equations 5 and 6, we have:

$$V_0 = \frac{bd}{1.5}\frac{\lambda\tau'}{(bdL)^{1/\kappa}} = \frac{\tau_u}{1.5}bd\frac{\lambda}{(bdL)^{1/\kappa}}\frac{\tau'}{\tau_u} \tag{7}$$

and
$$V_u = r_v V_0 = \frac{\tau_u}{1.5}bd\frac{r_v\lambda}{(bdL)^{1/\kappa}}\frac{\tau'}{\tau_u} \tag{8}$$

where V_u is the maximum shear, r_v is the ratio V_u/V_0 and τ_u is a reference shear stress for a chosen base volume. The base volume chosen for this section is 1.0 m³.

Equation 8 is expressed as:

$$V_u = \frac{\tau_u}{1.5}bd\,k_v \tag{9}$$

where:
$$k_v = \frac{\lambda r_v}{(bdL)^{0.18}}\frac{\tau'}{\tau_u} \tag{10}$$

is the size effect factor which corresponds with the modification factor for shear in the Code. The value of τ', identified as τ_u in Reference 15, is based on standard shear block tests as indicated in that reference, and adjusted as outlined in the commentary to Table 13 — 11.2(a). The value of τ_u is an adjusted value for size effect based on a volume 1.0 m³, i.e.:

$$\tau_u = \tau'\left(\frac{V}{V_1}\right)^{-1/5.53}$$

Thus $\tau_u = 0.14\,\tau'$, where $V = 1.0$ m³ which is 61 023 cu. in. and $V_1 = 1$ cu. in.
Substituting $\tau_u = 0.14\,\tau'$, Equation 10 becomes:

$$k_v = \frac{7.3\,\lambda r_v}{(V)^{0.18}} \tag{11}$$

where V is the volume in cu. in. When V is the volume in m³, the formula becomes:

$$k_v = \lambda r_v\,(V)^{-0.18}$$

The value of λ, identified as in Reference 13, can be tabulated for any load situation.

In the case of moving loads, the shear envelope determines λ. For moving loads on simple-span and continuous-span prismatic beams, the value λr_v may be conservatively taken as 1.72. The value of V_u is that found for the most severe position of the live load. When the dead load exceeds twice the live load, or when cases other than those covered in this section are encountered, a numerical integration may be used to determine λ. The procedure is as follows, and is based upon Reference 16.

a) Construct the envelope of maximum absolute shear force.
b) Divide the beam into segments of approximately constant shear.
c) For each segment, i, evaluate the maximum shear in the segment, V_{mi}, the dimensions b_i, d_i, and the segment length l_i. Calculate:

$$Q_i = 2\left(\frac{V_{mi}}{V_0}\right)^{5.53} \text{ and,} \tag{12}$$

d)
$$\lambda = 1.4\,C\left(\sum_{i=1}^{N} Q_i\frac{b_i d_i l_i}{V}\right)^{-0.18} \tag{13}$$

where V is the volume of beam in m³ and C is a correction factor. The theory as outlined in the reference above is based on a two-parameter Weibull probability model. Recent unpublished work by Foschi indicates that the more complex three-parameter Weibull model is appropriate, and a correction to the above for this latter can be expressed as:

$$C = 1.33V^{0.03}$$

When the correction is applied we have:

$$k_v = 1.72V^{-0.18}(1.33V^{0.03}) = 2.3V^{-0.15}$$

The correction $C = 1.33V^{0.03}$ should also be applied when using the numerical procedure.

For glued-laminated members with existing end splits, Equation 3 applies.

Compression Members

C13—8

Several changes from conventional North American practice appear in this clause. These changes have been introduced to provide a more rational approach to design and analysis of compression members. The changes which have been made are summarized below.

1/ Moments induced in the column are determined by a moment amplification procedure. An accurate method is specified in Clause 13—8.2.1 and an approximate method is given in Clause 13—8.4.1(b).

2/ The modulus of elasticity used in the determination of P_{cr} is E_{05}, the fifth percentile value.

3/ The end eccentricities for the load P have been defined to have a specified minimum value.

4/ An allowance is made for bow of compression members.

The general approach to design and analysis of compression members is nearly identical with that used in Reference 17, except for tapered round piles, where a new analysis is based on the results of an extensive computer study [18] and on the tolerances permitted in Reference 19.

Evaluation of Slenderness Effect

C13—8.3

C13—8.3.2

Effective Length

In determining L_u in conjunction with the base or cap detail, the distance should be measured from the outer surface of the base or cap plate. The effective length factor, k, should not be less than what would be indicated by rational analysis. Where conditions of end restraint cannot be evaluated accurately, a conservative value for k should be used.

The values of k in the last four cases in Table A13—1 can be greatly exceeded due to minor reduction in the degree of rotational restraint.

C13—8.3.3

Effective Length of Members Braced Against Side-Sway

The recommended value of k is 1 for compression members braced against side-sway and is in conformity with Reference 17. However, where determined by analysis or from Table A13—1, other values of k may be used.

C13—8.3.4

Effective Length of Members Not Braced Against Side-Sway

For members not braced against side-sway, the value of k may be determined by analysis or, alternatively, from Table A13—1 where applicable, but k shall not be less than unity.

C13—8.3.5

Conditional Limitations

Calculations have shown that when kL_u/r falls below the values in Clauses 13—8.3.5(a) and (b), the effect of moment amplification is not greater than 5% and may be neglected.

C13—8.3.6

Interaction for Uniaxial Bending

This is the common uniaxial bending interaction equation but uses the stub column capacity in the first term of the equation. The use of this equation yields capacities close to those obtained by the approach given in Reference 17, which differs from that used in steel design.

This interaction equation implies that the maximum extreme compression fibre stress controls.

The interaction equation formula ignores torsional buckling and is therefore only satisfactory for round or nearly square columns. If a rectangular column has a ratio of long-to-narrow dimensions of its cross-section which exceeds 1.25, the value of P_u should be computed using a slenderness reduction factor based on the smaller dimension. This factor might be obtained by a method such as that used in Reference 11 using L_e for bending perpendicular to the plane of principal bending.

Approximate Method for Compression Members

C13—8.4

C13—8.4.1

The column moment is determined by the approximate formula δM_0 rather than as $M_0 + P\Delta$ as arises from Clause 13—8.2.1. The error involved in this approach is acceptable in view of the uncertainties related to material properties, initial bow, and minimum end eccentricities.

C13—8.4.3

This is a conservative value for C_m.

End Moments

C13—8.5

C13—8.5.1

This is a new provision in North American wood compression member design. A similar provision has long been used in reinforced concrete design. It is almost impossible to apply an end load to a column with zero eccentricity. The minimum end eccentricity of 0.05 times the lateral dimension in the plane of bending being considered is one-half of that used for reinforced concrete columns [17], but in the latter case no allowance is made for compression member bow.

Bow Moments **C13—8.6**
The specified bow in this clause is approximately one-half of that permitted in sawn timber beams.

Biaxial Bending **C13—8.7**
This clause provides the interaction equation where biaxial bending exists. The designer must make his own determination of the critical combination of M_x and M_y.

Slenderness Effects for Piles **C13—8.8**

C13—8.8.1
The procedure used for round piles is similar to that used for other compression members in that the same basic moment amplification procedure is to be used. However, M_0 is replaced by M_p and P_{cr} is determined by a formula which takes account of the pile taper and shape of cross-section.

Table A13—3 gives values of η to be used for various values of k_u and various combinations of tip and butt dimensions. The η values were determined by numerical analysis [18].

Effective Length of Piles **C13—8.8.2**
The effective length of a pile has been taken as defined in Reference 20.

C13—8.8.3
This clause defines M_p and the location of the cross-section at which it occurs. The critical section location was determined by computer analysis and is a good approximation to the exact location. The e_0 values of Table A13—2 were determined by a probabilistic simulation based on the limits of bow and crook given in Reference 19.

Compression or Bearing Not Parallel to the Grain **C13—10**

Loading at an Angle to the Grain **C13—10.2**
The familiar Hankinson format [11] has been modified to include specified strengths, duration of load, load sharing, and the performance factor. For loads at an angle to the grain ϕ has been set at 0.9, as for compression parallel to the grain.

Size Effect **C13—10.3**
The modifications for location and length of bearing are essentially those used in Reference 11.

Sawn Wood **C13—11**

Materials **C13—11.1**

C13—11.1.1
Species and Species Combinations

There is an increasing tendency for the lumber industry to market structurally-graded dimension lumber, 38 to 89 mm in thickness, by species combination rather than by individual species. The grade stamps of such species combinations will bear designations such as "Hem-Fir", "S-P-F", etc. Lumber marked with such a combination stamp may be of one species only or any mix in the combination.

This should be borne in mind in the case of permanent structures where pressure preservative treatment is required, as not all species in a particular combination are readily treatable. The pressure preservative treater must cull out from a species combination those species which are not treatable.

Individual species and treatable species combinations should be specified for permanent structures. In the case of large timbers, individual species generally are available. Species combinations, not individual species, should be specified for untreated temporary structures.

Names of species in Table 13—11.1.1 are standard commercial names. Additional information on botanical names and other common names is given in Reference 21.

Some of the stamp identifications listed in Table 13—11.1.1 contain the letter N in brackets. This is required for lumber shipped to the United States to distinguish between similar American species. It will often be found on Canadian lumber.

C13—11.1.2
Grades of Sawn Wood

The design methods and data in this section apply only to softwood dimension lumber and sawn timbers that have been graded and identified in accordance with NLGA grading rules [22].

Although hardwood often has been used in temporary structures, this section makes no provision for hardwood. Reference 11 provides a technique for the use of hardwood, if it is regarded as if it were softwood and grade marked in accordance with Reference 22. However, because of the unavailability of grade-marked hardwood and an inadequate data base for material properties, hardwoods are not permitted in this Code.

C13—11.1.3 Identification of Wood

This clause requires that all wood in both temporary and permanent structures be grade marked.

Where an individual species is supplied by a pressure preservative treater, the stamp may identify the species or it may have a combination stamp. Further preservative treatment may obscure the grade stamp. In these cases, evidence should be obtained as to what species was treated.

Specified Strength and Modulus of Elasticity C13—11.2

The specified bending strengths for 38 mm dimension lumber have been developed from a testing program with full-size members carried out by B. Madsen [23]. Statistical analyses were performed by Bury [24]. The contents of these reports were summarized by NLGA in Reference 25.

Specified strengths for timbers are based upon full-size testing reported in Reference 26. Additional tests on 89 mm dimension lumber show strength values comparable with posts and beams. Therefore, the specified strengths for 89 mm dimension lumber are to be taken as specified for posts and beams. Tension values have been based upon information contained in References 4 and 5. Compression values have been based upon information contained in Reference 27. In all cases, tests were carried out on full-size members graded in strict accordance with the NLGA grading rules.

The above information has been collected for the three major species groups: Douglas Fir-Larch; Hem-Fir; Spruce-Pine-Fir. Values for other, less-used species groups have been conservatively assessed, using judgement.

The use of data from testing with full-size members for determining strength properties is a departure from present North American practice where the ASTM method using small clear specimens has formed the basis for developing allowable stresses. The new method (referred to as "in-grade testing") has shown that some of the effects observed in small clear specimen testing were being masked by the presence of natural growth characteristics allowed in commercial lumber. This manifests itself in three areas.

i) A substantial size effect is found. Small depth beams give higher strength values than larger depths for the same grade.
ii) The grading rules do not provide material in which the strength of No. 1 grade is significantly different from the strength of No. 2 grade. In this Code, these two grades have been given the same values.
iii) The strength differences at the 5th percentile between species are much smaller than previously determined from small clear tests.

In addition, it has been observed that the effect of moisture content upon strength is dependent upon the strength of the material. Thus, very strong material is more sensitive to moisture content than the weaker material.

The above deviations from traditionally-held opinions can, to a large extent, be explained by the fact that the failure mode for clear material is essentially a compression failure, while the failure mode of commercial material (at the 5th percentile) is a brittle failure associated with local slope of grain around knots and other natural growth characteristics.

The values in the Tables 13—11.2(a) and (b) represent 5th percentile estimates of short-term strength, reduced to 85% to allow for duration of load (one month) and normalized to a 286 mm depth.

These values should then be adjusted by the designer in cases where the duration of load is different or where the depth of the member is different from 286 mm. No adjustment should be made for moisture content.

The degree of confidence in the data is reflected in the performance factor, as discussed in Commentary Clause C13—4.4, not in these specified strength values.

The specified strength values for shear were determined in the manner outlined in Commentary Clause C13—12.2 for glued-laminated timber.

Structural Glued-Laminated Timber C13—12

Materials C13—12.1

C13—12.1.1

Material fabricated under existing CSA standards has been tested to support the previous allowable stresses. The specified strengths herein have been derived from these previous stresses except as noted below in Commentary Clause C13—12.2. The details of fabrication and quality assurance are found in References 28 and 29.

Specified Strength and Modulus of Elasticity **C13—12.2**

Calibration studies indicate that the load effects of this Code are approximately 1.8 times those encountered under previous design standards [20]. It is anticipated that actual loadings may be approximately 10% greater in Ontario than those assumed for previous design standards, but this is overshadowed by the fact that the current Code requires that impact be considered. Thus, the specified strengths listed in Table 13—12.2 are approximately 1.8 times those provided in Reference 11, except in the case of shear.

Longitudinal shear values are based on the mean values for shear parallel to grain given in Reference 30, adjusted as follows.

a) A reduction of 10% was applied to dry values to correct for moisture.

b) The mean values were adjusted to represent 5th percentile values and corrected for size effect to a volume of 1 cu. in. using the expression from Reference 13:

$$\tau'_{0\,05} = 1.95\tau_{ASTM}\,[\ln 0.95]^{1/5.53}$$

c) The values $\tau'_{0\,05}$ were multiplied by $(61\,024)^{-1/5.53} = 0.14$ to correct to a base volume of 61 024 cu. in. = 1.0 m^3.

d) Values were multiplied by 0.85 to correct to a load duration of one month and rounded to ± 5%.

The values for tension are not well supported by tests, and some recent testing indicates a possible volume effect (see Commentary Clause C13—5.4).

The values of E_{50} are taken directly from Reference 11. Current practice includes rating the laminating stock for stiffness. The values of E are therefore supported by excellent control. The values of E_{05} are determined by assuming that the coefficient of variation of E is approximately 10%.

Vertically-Laminated Beams **C13—12.3**

This clause was included to differentiate between beams made of horizontal laminations and those made of vertical laminations.

Camber **C13—12.4**

The provision for camber includes twice the dead load deflection, in order to allow for long-term creep. The additional camber of 1/600 of the span provides a margin against the unsightly appearance of a sagging member. The rare occurrence of full live load does not justify a design requirement for this loading.

Varying Depth **C13—12.5**

Varying depth causes some tension perpendicular to the grain and can be dangerous. The bevel is limited so as to control the effective inclination of the grain to the tension trajectories, which are parallel to the surface. Bevel is not permitted when the tension values are high.

Curved Members **C13—12.6**

The minimum radius of 12 m has been selected in order that the following effects shall be of no consequence:

a) the bending stress in the individual laminations during lay-up;

b) the tension perpendicular to the grain when a curved member is subject to bending inducing a change in curvature opposite to the unloaded curvature;

c) the departure from simple bending theory when dealing with curved beams.

For special cases, Reference 11 may be consulted.

Wood Piling **C13—13**

This clause is closely patterned after Reference 11 except that the design of the unsupported length is to be in accordance with the procedure of Clause 13—8 dealing with compression members.

Splicing **C13—13.2**

The Ministry has experienced difficulty in maintaining quality control in field splicing. Its use is being discouraged.

Specified Strength and Modulus of Elasticity **C13—13.3**

The specified bending strength and the values of E_{05} were derived from extensive data on tests of timber poles in bending as compiled in Reference 31. The specified compression strength has been taken to be the same as the specified bending strength.

Design **C13—13.4**

C13—13.4.2 **Embedded Portion**

Soil, unless it is extremely soft, provides sufficient lateral support to permit the pile within the soil to be designed as a short column. For ease of calculation, the maximum load is assumed to act on the minimum cross-section at the tip for end-bearing piles.

For a friction pile, in order to account for some load transfer to the soil above the tip, the maximum load may be assumed to act on the cross-sectional area, one-third of the thickness of the supporting stratum above the tip of the pile.

C13—13.4.3 **Unembedded Portion**

Where adequate bracing is not provided, the unsupported length of the column (generally that section above soil support) is assumed to be hinged at the point of contraflexure in the soil and at the pile cap. The pile is to be designed as a tapered column taking into account the butt eccentricity, plus any eccentricities permitted in Reference 19, as well as the effects of any lateral loading.

Fastenings **C13 — 14**

Materials **C13 — 14.1**

C13 — 14.1.3 Permanent Structures

Glulam rivets and truss plates have been excluded because of corrosion, except that steel nail plates are permitted in Clause 13 — 21.2.1.3.

Specified Strength **C13 — 14.2**

There are not sufficient satisfactory test data available on which ultimate strengths of fastenings can be based. As a result, the specified strengths tabulated were rationalized as follows.

The design load in this Code is based on a survey of truck loads currently on the Ontario highways. The joints in the present structures are not exhibiting distress due to this loading. A study was made comparing the loading effects of the factored OHBD Trucks to the H20S16 loading over a range of spans from 6 to 36 m. The results of this study showed that the force effects of the factored OHBD Truck were from 1.4 to 1.8 times those of the AASHTO loading. This information, along with experience and the examination of available test data, resulted in the selection of a factor of 1.7. The tabulated values in Reference 11 are for dry service conditions and must be modified by the requirements for wet service conditions.

Hardware and Metal Work **C13 — 15**

Protective Treatment **C13 — 15.2**

Ministry experience has shown that it is both necessary and justified economically to hot-dip galvanize all hardware. Heat-treated alloy bolts will be affected by the heat of the zinc bath; therefore, some other method of protection, such as zinc-rich paint, must be employed.

Pressure Preservative Treatment **C13 — 16**

General **C13 — 16.1**

Although the initial cost is higher for pressure preservative treated wood, the cost for a permanent structure is fully justified on an annual cost basis by the extended life and the assurance of continued performance.

The two preservatives listed are considered to provide permanent treatments. Design and construction documents should not simply refer to Reference 32 but should also specify the preservative that is to be used, since that standard also covers a number of other preservatives. Except as provided for in Clause 13 — 16.2, the water-borne preservatives, CCA and ACA are not acceptable under this Code due to extensive splitting that has been reported following treatment with these preservatives. Further, although CCA and ACA are generally considered to be non-leachable, there is a potential for galvanic corrosion of hot-dipped galvanized nails, which are called for in nailed-laminated decks. Pending further investigation into the potential for galvanic corrosion and the reportedly reduced effects of splitting and checking in smaller sized materials, which can be kiln dried after treatment, consideration may be given to the use of CCA and/or ACA for dimension lumber.

Pedestrian Contact **C13 — 16.2**

Main structural members must be treated in accordance with Clause 13 — 16.1 due to the problems associated with the use of CCA and ACA described in Commentary Clause C13 — 16.1. Since creosote causes severe skin irritation and pentachlorophenol is regarded as hazardous under repeated human contact [33], it is necessary to prevent direct pedestrian contact with main structural members. Only those components other than main structural members which are subject to direct pedestrian contact should be treated with CCA or ACA.

Untreated Piles **C13 — 16.3**

Decay may occur in continuously submerged piling exposed to flowing water [34, 35, 36, 37]. The three criteria listed must be satisfied to permit the use of untreated piling.

Incising **C13 — 16.4**

Preservatives travel more readily along the grain than across the grain. To assist in obtaining deeper and more uniform penetration it is common practice to incise wood before treating, i.e., the lateral surfaces are opened up by knife-like incisions, parallel to the grain, of a uniform pattern and depth.

Fabrication **C13 — 16.5**

Field cutting into pressure-treated wood with the potential reduction of protection should be kept to a minimum. Hence, plans should provide as much detail as possible to facilitate pre-framing prior to preservative treatment. There are, however, field situations such as drilling holes for bracing which may not be avoidable. These cuts and other field damages should be field-treated by three thorough soakings, each separated by an adequate interval of time, with a field cut preservative suitable for the preservative used in the original pressure treatment. Spraying is not acceptable. Information should be conveyed to field personnel by means of plans or other appropriate methods to ensure that they are informed about the proper handling and aftercare of preservative treated wood.

Wood Cribs

C13—17

General

C13—17.1

C13—17.1.3 Experience has indicated that closed-faced cribs eliminate some of the problems that can be caused by ice and erosion.

Member Sizes and Assembly

C13—17.2

C13—17.2.1 Members less than 184 mm have not proven robust enough in service.

Bearings

C13—17.4 Experience has shown that rotation of the crib and/or deflection of the supported structure has resulted in eccentric loading of both. This clause has been written so that a bearing will be provided to minimize such eccentricity.

Wood Trestles

C13—18

General

C13—18.1 This part of the section has been written to allow the designer more freedom of choice in the areas of piling, posts, and bracing than was available in previous codes. Previous codes specified the following minimum requirements which may be used for guidance.

- A minimum of 5 piles or 6 posts per bent to be provided, with the outside members battered 1:8.
- Single-storey longitudinal bracing is to be provided for bents between 3.5 m and 6 m in height, and multiple-storey bracing over 6 m.
- At least every other bent is required to be braced with not less than two pairs of diagonal braces and girts bolted to the posts or piles next to the outer posts or piles.
- For greater efficiency it is recommended that the slope of the diagonal bracing should not exceed 45° with the horizontal.

The tops of piles connected to pile caps by means of drift pins tend to split due to lateral loads. The steel collar is intended to prevent this.

Framed Bents

C13—18.3

C13—18.3.3 **Post Connections**

This clause specifies the minimum permissible connections. Stronger connections may be required in order to meet the buoyancy requirements of Section 2.

Stringers and Girders

C13—19

Diaphragms

C13—19.2 It is advantageous to have diaphragms as deep as practicable and with support diaphragms located approximately one beam depth from the support. Cross frames nailed to wood stringers or girders have been found to be ineffective and should be avoided.

C13—19.2.2 These requirements have been specified to ensure stability of the beams.

Nailed-Laminated Wood Decks

C13—20

General

C13—20.1 This clause covers only those laminated wood decks which may be constructed by nailing.

Transversely-Laminated Wood Decks

C13—20.2 The lamination dimensions are compatible with the requirements and pattern specified. Thicker laminations would require considerably larger nails. Smaller lamination widths may introduce incompatible edge distances for the nails used. To avoid the reflection of irregularities in the asphalt surface, a tolerance of 5 mm has been specified.

C13—20.2.1 **Assembly**

C13—20.2.1.1 **Nailing**

The method of fabrication specified by this clause has been used for many years and has proven adequate.

C13—20.2.1.2 and C13—20.2.1.3 **Deck Support Anchorages**

The laminations should be securely fastened to the longitudinal beams to ensure satisfactory performance. The methods specified have been used for many years and have proven adequate.

C13 – 20.2.1.4 Providing a tight deck will prevent undue access of deleterious materials and excessive lateral load on the nails. It is important to ensure that the decking transmits loads by direct bearing and that uneven seating does not occur at the bearing points. This reduces undesirable movements under loading which can severely damage the deck system.

C13 – 20.2.1.5 **Butt Joints**

Butt joints are to be distributed to ensure reasonable continuity of the deck. Butt joints in the vicinity of the wheel paths should be minimized as they tend to reduce the service life of the deck.

Longitudinally-Laminated Wood Decks

C13 – 20.3 For longitudinally-laminated decks, the conventional method of nailing laminations together, as specified for transverse laminated decks, has not proven to be completely dependable under continuous service. The action of repetitive loading works the nails loose, causing the laminations to separate and resulting in independent movement of the laminations. This system poses severe maintenance problems early in the service life of the deck.

Wood-Concrete Composite Decks

C13 – 21

General

C13 – 21.1 Wood-concrete composite decks have been built in Ontario for many years and have displayed a good record of service.

Wood Base

C13 – 21.2

C13 – 21.2.1 **Assembly**

C13 – 21.2.1.3 **Spliced Butt Joints**

The galvanized splice plates are provided to ensure integrity of the deck system under the action of repeated service loads.

C13 – 21.2.1.4 **Deck Anchorage**

The method specified has been used for many years and has proven adequate.

Concrete Slab

C13 – 21.3

C13 – 21.3.2 **Thickness**

The net thickness is the thickness of the concrete above the higher lamination. This minimum value ensures adequate cover over the reinforcement.

C13 – 21.3.3 **Reinforcement**

The reinforcement specified is provided to resist the effects of shrinkage and temperature and to ensure adequate load distribution.

Wood-Concrete Interface

C13 – 21.4

C13 – 21.4.2

a) The requirements of this clause are based upon existing practice and have been used successfully since 1955 [38].

b) This design is based upon recent research and development [39, 40].

Factored Moment Resistance

C13 – 21.5

C13 – 21.5.1 **General**

The determination of resistances assumes that the materials are linearly elastic. The determination of the specified moduli of elasticity is in contrast to the conventional values previously used in which the E for concrete was a prescribed multiple of E for wood, dependent upon the thickness of the components.

C13 – 21.5.2 **Factored Positive Moment Resistance**

This approach takes into consideration the effects of redistribution of dead load forces due to creep. Therefore, the composite section is assumed to carry both the dead load and live load.

Prestressed Wood Decks

C13 – 22

General

C13 – 22.1 As a result of developing a method to rehabilitate existing longitudinally-laminated nailed decks, a method of prestressing was developed [41]. Subsequent research [9, 42, 43, 44, 45, 46, 47, 48, 49] further developed this method of prestressing wood decks so that it could be applied to the design of new bridges.

A prestressed wood deck is a longitudinally-laminated wood deck in which the prestress is applied perpendicular to the laminations. This pressure creates friction between the laminate surfaces which, in effect, replaces the need for nailing. The resulting orthotropic deck slab has load distribution qualities far superior to the conventional nailed decking, is more durable under the application of repetitive loadings, and displays higher strength as a result of load sharing.

Prestressing Materials C13—22.2

C13—22.2.1 **Prestressing Steel**

Environmental protection for the high-strength steel used in prestressing these decks has been given considerable importance. Consistent with this, only bar size material is being allowed, which is considered to have better resistance to corrosion than either cables or strands. Adherence to the specifications in Reference 50 ensures that the quality of steel will not be less than that allowed in the design of prestressed concrete.

C13—22.2.2 **Anchorages**

Anchorages should not contribute to prestress loss after transfer. They should be easily accessible for restressing and should be able to withstand up to six restressing procedures (and hence relocation along the bar) without reducing their capacity or performance.

C13—22.2.4 **Stress Limitations**

Due to the large immediate prestress losses that occur in a prestressed wood deck, and since the bars are not used as direct load-carrying elements, higher initial jacking forces are allowed than those used in prestressed concrete design.

C13—22.3 **Design of Prestressing System**

The external system was developed to rehabilitate existing decks where it is not practical to locate bars within the wood deck.

The internal system is recommended for new construction as it removes obstruction from the surface of the deck and provides much better protection against mechanical damage.

C13—22.3.1 **Steel/Wood Ratio**

One of the most important criteria in the design of a prestress system is to provide enough flexibility to offset the prestress losses due to the effects of creep in the wood. Considerable research has been performed [41, 43, 47] to produce design requirements which will control prestress loss. In order to simplify the design, avoiding complicated stiffness calculations, the flexibility limitation is imposed by specifying a maximum steel/wood ratio.

C13—22.3.2 **Distributed Normal Pressure on Laminations**

Increasing the initial pressure beyond this level does not significantly increase the final pressure level after losses have occurred.

Laboratory studies [43, 47] have indicated that prestress loss is not likely to exceed 50% as long as all other design specifications are followed. Field observations by the Ministry [41] have indicated a much smaller loss.

C13—22.3.3 **Stressing Procedure**

The specified stressing sequence is based upon laboratory studies [43, 47]. Restressing too soon will lead to larger losses. However, increasing the time interval between stressing cycles will be beneficial, except that little additional benefit is obtained when extending Stage 2 beyond six weeks and Stage 3 beyond six months.

Design of Distribution Bulkhead C13—22.4

C13—22.4.1 **General**

Distribution bulkheads are required along the edges of the deck to distribute the prestressing forces to the wood. Ministry experience has shown that discontinuous distribution elements do not perform well and invariably rotate causing local crushing of the wood deck.

C13—22.4.2 **Factored Bearing Resistance to Prestressing**

Some slight local crushing is unavoidable due to the flexibility of wood. However, this will not be readily visible.

C13—22.4.3 **Bearing Area for Prestress Force**

In order to satisfy Clause 13—22.4.2, it is necessary to determine the area over which the prestressing force is distributed directly to the external wood laminate. For designs other than that prescribed under Clause 13—22.4.4, the designer must determine this area; however, some direction may be taken from Commentary Clause 13—22.4.4.4.

C13 — 22.4.4 **Steel Channel Bulkhead**

The steel channel bulkhead specified has already been successfully used on several bridges in Ontario, however, the designer may choose to use any bulkhead design which satisfies the requirements of Clauses 13 — 22.4.1 through 13 — 22.4.3.

C13 — 22.4.4.1 **Channel**

A heavy steel channel has been specified to ensure adequate stiffness at jacking.

C13 — 22.4.4.2 **Anchorage Plate**

The plate proportions specified ensure adequate stiffness.

C13 — 22.4.4.3 **Spacing of Prestressing Stations**

The spacing corresponds to the channel sizes specified.

C13 — 22.4.4.4 **Effective Bearing Area**

The prestressing forces are assumed to be uniformily distributed over the specified areas. This assumes a 45° distribution through the channel.

C13 — 22.4.4.5 **Web Stiffening**

The systems shown in Figure 13 — 22.4.4.1(a) and (b) ensure reasonably uniform pressure over the effective bearing area. Alternative designs must provide similar performance.

The method given has been used several times in the past and has proven adequate.

Laminated Deck **C13 — 22.5**

C13 — 22.5.2 **Lamination Dimensions**

The provisions of this clause are to ensure that the load-sharing factor given in Clause 13 — 5.6 is applicable.

C13 — 22.5.4 **Nailing**

The nailing specified is for construction purposes and must be provided. Prior to prestressing the deck the effects of construction loads should not exceed those due to 25% of the unfactored OHBDC truck.

C13 — 22.5.5 **Support Anchorage**

The decking will move during the prestressing operations and any restraint against this movement may be detrimental to either the restraint being used or the prestress level in the deck.

Factored Moment Resistance **C13 — 22.6**

The determination of resistance assumes that the wood performs linear elastically.

C13 — 22.6.1 **Net Section**

Except in cases where the deck thickness is controlled by deflection, it will probably be advantageous to restrict the use of the butt joint pattern permitted in Clause 13 — 22.5.1 to areas where the moment does not exceed three-quarters of the maximum moment. When deflection does govern the deck thickness, butt joints should be distributed throughout the deck to enable the design to maximize the use of "as supplied" stock lengths.

Factored Transverse Shear Resistance **C13 — 22.7**

It is necessary to ensure that the friction between the laminations, induced by prestressing, is capable of transferring the effects of the factored loads to laminations adjacent to those which are directly loaded. The specified friction coefficients given in Table 13 — 22.7 were derived from laboratory research [43, 44].

It is not necessary to check longitudinal shear.

Environmental Protection **C13 — 22.8**

C13 — 22.8.1 **Hardware**

The anchorages and bars must be protected against deterioration.

The method specified has been used in Ontario although it has only been in service a few years.

C13 — 22.8.2 **Preservative Treatment for Deck**

This treatment provides the best known resistance to variations in humidity which can cause volumetric changes in the wood. Further research is being performed to determine the effects of various preservative treatments.

Design Details **C13 — 22.9**

C13 — 22.9.1 **Curbs and Guiderails**

The guiderail should not be capable of transmitting impact forces directly to the steel bulkhead. This will reduce the chance of failure of a prestressing bar in the event of a vehicular collision with the guiderail.

C13 — 22.9.2 **Prestress Failure Protection**

In the unlikely event that a prestressing bar fails, the steel bulkhead should be provided with a method to restrain the prestressing components. The restraining device should be designed so that it can be removed easily to provide access to the anchorages.

Wearing Surface C13—23

General **C13—23.1**

The shell of preservative-treated wood must be protected from mechanical abrasion. All of the methods specified have been used successfully in the past.

Plant-Mix Asphalt **C13—23.2**

Thin layers of asphalt may break up on a nailed wood deck. The use of welded wire fabric to reinforce the asphalt and prevent cracking has proven very effective in the past. Prestressed wood decks do not require reinforcement in the asphalt as they display virtually no relative movement of the laminations under loads. Spraying the wood deck surface with an asphalt primer prior to applying the asphalt surface has proven to increase the bond between the two materials.

Drainage C13—24

General **C13—24.1**

Water can have a deleterious effect on the structural integrity of a wood bridge in many areas. Water can cause repetitive volumetric changes which can loosen joints, can soften the wood surface, and may initiate decay. It is extremely important that adequate provision be made to conduct water away from all parts of the bridge. All joints, connections and other details should be designed to minimize the entrapment of moisture.

Requirements For Decks **C13—24.2**

The curbs are to be raised higher than was previously required in an effort to reduce the problems associated with foreign materials blocking the flow of water.

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C14 Evaluation of Existing Bridges

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Scope

C14—1

The provisions of this section are applicable only to those bridges which have proven themselves capable of sustaining wind, ice forces, and other environmental loads. Serviceability limit states are not considered except as noted within Section 14 and commented upon in Clause C14—6.2.

Soil-steel structures free of distress or deterioration, and with a satisfactory performance over a period of several years, will normally not require evaluation.

Masonry bridges have not been included due to the lack of information in limit states format. References 1 and 2 are recommended for information on the design of masonry bridges using the working stress method.

Definitions

C14—2

Controlled Vehicle — a controlled vehicle, for the purpose of this Code, is a vehicle with a gross load in excess of the posted or legal loads and where the axle groups are not necessarily in conformity with the Highway Traffic Act. An example would be a trailer with twelve axles of eight wheels per axle and a gross vehicle mass of 123 000 kg. This is not a vehicle travelling under an Annual Permit or travelling under a Normal Single-Trip Permit as issued by the Drivers and Vehicles Branch of the MTC or other road authorities. A detailed engineering investigation of all bridges on the route would be necessary before the issuance of a Special Single-Trip Permit for the vehicle.

Routine Inspection — the present Ministry policy requires that every bridge within its system must be inspected every two years by engineers or technicians.

Secondary Components — examples are wind bracing and some intermediate diaphragms.

Need for Evaluation

C14—4

a) Bridges deteriorate with time and may be damaged by vehicles, storms or floods. They should be inspected by an engineer following any structural changes and should be evaluated to determine the load-carrying capacity.

b) An engineer should evaluate the bridge and determine its load capacity prior to a load-restricting by-law being enacted.

   A bridge which has been previously evaluated or posted as a result of an inspection and structural evaluation may not require a recalculation of the load capacity unless the road classification has changed and/or the re-evaluation inspection indicates that further deterioration or distress has occurred.

c) An evaluation of the load-carrying capacity of a bridge is recommended by the Ministry before a bridge is scheduled for replacement or rehabilitation.

d) The movement of heavy commodities such as hydro transformers, large off-road vehicles, and boats will usually necessitate that all bridges on the specified route be evaluated to determine their load capacity and whether they should be strengthened.

Condition Survey

C14—5

The main goal of a condition survey is to determine if the structural assemblage and its individual components are responding to past and present loads in an acceptable manner and to extrapolate these results for a reasonable period of time into the future. In order to achieve this goal, two distinct streams of activity are necessary — both dealing with applied loads and structural response mechanisms. One stream consists of the theoretical or experimental analysis of the behaviour. The second stream is the condition survey — the physical evidence of the bridge's geometry, condition, and response to loads. A synthesis of the individual component state and condition in conjunction with analytical investigations will usually determine the structure's behaviour and safe load-carrying capacity.

General

C14—5.1

The practice of condition surveying entails the detailed inspection of all components, their connections, and their assemblage.

In undertaking routine bridge inspections, it is usually satisfactory to use judgement with respect to alignment survey, deformation, etc., by lining one component up against the next or sighting along the members from a point near the end of the member or by utilizing binoculars. When visual inspection is not sufficient, detailed measurements of the structure geometry are necessary. For this type of condition survey, access must be made available so that the inspector can touch and sound all parts of every component in the structure.

Documentation should be available and include a complete component listing and classification and an inventory of all initial alignment measurements, geometry, initial defects, and any rehabilitation, retrofitting, replacement and widening details. If available plans do not represent actual features, it is desirable that plans should be updated or, if not available, they should be prepared.

Special attention should be paid to alignment and permanent deformation, especially with respect to arches, columns, and other primary components predominantly in compression. Deformations in such members can result in loss of load-carrying capacity.

The documentation of all inspection data including initial defects, deterioration, section loss, and material strengths, is extremely important in order that quantitative and qualitative comparisons of change can be made. Note that change in response should be emphasized as the absolute value of deformation, cracking, or loss of section may not be meaningful in itself without reference to the past data. The engineer must use judgement to determine what physical evidence is indicative of component failure or the inability of the component to perform in an acceptable manner.

In appraising the results of a condition survey, it is necessary to consider the importance of the component under study as it relates to the load-sharing system.

It is usually found that those components least important for strength characteristics are of the utmost importance with respect to durability. An example is the expansion joint, which, if not functioning correctly, can cause deterioration and failure of primary components such as girders.

The net component section available for load sharing must be determined by measuring the section loss. Any loss of section due to the environment, accident damage, salt, or other contaminants, must be taken into consideration in generating the component section properties. Examples of deterioration of components include spalling of concrete, delamination of concrete, deterioration of prestressing steel, excessive rusting and pitting of mild or weathering structural steel, corrosion of rebar, fungus attack of timber, and crystallization of rubber at low temperatures.

The condition of connections and component boundaries can often lead to information that can give guidance with respect to behaviour of the structures.

Material strengths taken from plans are usually based on minimum requirements. The use of physical testing methods such as concrete coring and steel coupon tests will often yield material strengths much in excess of those anticipated. An increase in factored resistance, and accordingly in the net available live-load capacity, may result.

Components of different materials should be inspected with their particular properties and characteristics in mind. The approach should be planned, performed, and appraised utilizing all the current technology available with respect to the particular material. Some of the more common techniques are referenced in Code and Commentary Sections 14 — 5.2 to 14 — 5.5, inclusive.

References 3, 4, 5, 6, and 7 represent current bridge inspection and evaluation practices of a general nature and may be useful for undertaking condition surveys. In addition, many inspection manuals have been prepared by individual states in the USA, and a maintenance manual is being prepared by the Ministry.

Steel and Iron C14 — 5.2

The inherent characteristics of steel and historical fabrication methods have resulted in the use of relatively thin plates with stiffeners, splices, and other details which are susceptible to stress concentrations and fatigue. Chronological development from arch truss to truss, lattice girder, riveted plate girder, welded girder, and welded frame has introduced corresponding changes in connection stiffness. Additional joint stiffness has resulted in multi-dimensional strain distribution in connections.

The connections which are prone to fatigue cracking have been undergoing rapid design changes as a consequence of recent fatigue investigation. Fabrication could also play an important part in structural steel's susceptibility to fatigue crack growth. Micro-crack growth depends on the design detail, stress range and steel notch toughness, and could result in brittle fracture.

C14 — 5.2.1 Inspection of Components

A rational approach to component inspection is to assess the intent of the design and examine any physical evidence relevant to the expected responses.

In tension members, paint flaking or necking down could indicate that the yield point has been exceeded. Strain hardening or load redistribution might possibly have alleviated the problem.

In built-up compression members, imperfections or rippling of plates may lead to local failure in a portion of the member. Rippling of a cover plate may be initiated by rust and volume change at the faying surface.

In flexural members, careful attention should be paid to member or component alignment, e.g., excessive sweep, permanent deflection, or localized deformation. The recent requirement for curved plan geometry and aesthetics has resulted in the increased use of closed, torsionally-stiff box or trapezoidal sections. Special attention should be paid to stiffened bearing diaphragms of these systems. In any stiffened panels, a close examination of possible diagonal rippling from the compression panel corners (tension field) may be indicative of changing principal stress patterns and the onset of failure in panel.

In all steel components, discolouration, surface roughness, and paint flaking can be indicative of stress concentration in that particular area, making documentation of such items important.

The use of relatively thin steel plates and stiffeners results in abrupt discontinuities along and through component sections. Inspectors should recognise details that might generate fatigue cracking at fairly low stresses, including:

- the ends of welded cover plates;
- that area of the web near the gap between vertical stiffeners and flanges;
- re-entrant corners or copes;
- fabrication gouges;
- butt welds;
- secondary flange attachments; and
- cantilever bracket tie-back plates and the adjacent parts of main girders.

The use of specialized radiographic, ultrasonic, or magnetic particle testing methods is usually advisable at suspect details. Caution should be taken near, and at all connections — especially where special welding techniques have been used.

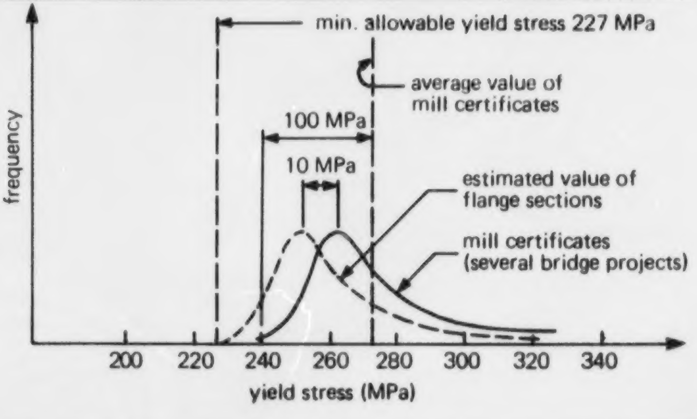
Steel bolted or riveted connections have historically been overdesigned; however, these are often in locations where deterioration is more prevalent. Bolt slip and rivet head microcracking should be checked. A rupture of the paint film and rust lines at faying surfaces are telltale signs of these phenomena.

Many relatively inexpensive techniques of actual physical testing of materials have been used, including the plaster casting of deteriorated surfaces, direct measurements by depth micrometer, calliper or straight-edge referencing, and dye penetrant techniques. Tension load may be quickly assessed by sounding or vibration techniques.

C14—5.2.2 Strength Properties

The strength of steel as related to yield point depends partly on the thickness of material, with the thinner elements having higher yield points. Thus, the steel in the webs of rolled beams has a higher yield point than that in the flanges. Samples taken from the thinner elements would indicate a value higher than the minimum yield point of the component. It is standard practice to take mill certificate coupons from the thinnest elements of a member, the resulting yield points of the samples being generally well above the minimum guaranteed yield points for any particular grade of steel. Figure C14—5.2.2 gives the results of test coupons taken from steel used in several bridge projects. Individual test results normally are within 10% of the average value for a given steel; hence results which differ from the average by more than 15% must be questioned.

Figure C14—5.2.2 Typical Distribution Curve for A-7 Steel



The values in Table 14 — 5.2.2 are adopted from discussions with Canadian steel companies and from References 6 and 8. Values given have been "soft-converted" to SI metric units. When a bridge is composed of different structural sections, such as channels, angles and I-beams, the engineer should not assume that all sections are of the same grade steel.

Concrete C14 — 5.3

The difference in the stress-strain relationship in tension and compression in concrete has disadvantages in terms of the low tensile strength, but provides advantages when undertaking engineering assessments. This is because of the visible distress of cracking in areas of principal tensile stresses in excess of the modulus of rupture.

A survey of significant structural cracking in concrete usually can result in a fairly accurate appreciation of the strain flow in a component, and hence of its behaviour. Care must be taken, however, in analysing the location and effect of reinforcement detail as it relates to cracking.

C14 — 5.3.1 Inspection of Components

Alignment survey and geometric properties are important in concrete component surveys, but with a much greater allowable tolerance than in steel components. Care should be taken in assessing permanent deflection and deformation since, in many cases, falsework settlement has resulted in permanent soffit curvature. An alignment check of fascia or deck slabs can often substantiate these initial construction deflections.

For concrete structures with monolithic framed connections, the significance of permanent beam deflections can often be verified by a check for cracking at the member boundaries.

Most concrete distress, apart from that due to improper alignment, can be classified as spalling, scaling, delamination or cracking.

Cracking in concrete, if of significant structural importance, should be surveyed and crack location, orientation, length, relative width and depth in the component should be documented. The age of the crack and its rate of change can be extremely helpful in categorizing crack type and appraising its cause.

Care should be taken in monitoring crack width and propagation as thermal loads will induce small fluctuations in all cracks. Periodic surveys will indicate these thermal movements to be a deviation of the major crack width measurements.

The type of crack (i.e., shrinkage, inclined flexure, torsion, or diagonal tension) can be determined in conjunction with an analytical appraisal of principal stresses and the tensile stress and orientation across the crack at the particular section under study.

The extent of loose or spalled concrete areas should be established, the areas of delamination, voids, exposed rebar, and effluorescence should also be noted. This type of distress is important with respect to durability considerations.

Extreme care should be taken when cracking and spalling occur together in compression zones. This is indicative of concrete crushing or bursting, and the onset of compression or shear failure (i.e., the anchorages of post-tensioned members or beam shear).

The deterioration of concrete decks as a consequence of the use of de-icing chemicals has been well documented [9, 10]. It is recommended that these decks be inspected by hammer sounding, chain drag, or other methods for signs of delamination. Half-cell potentiometer and pachometer readings in combination with the aforementioned, are usually necessary for rehabilitation design and durability considerations. It is recommended that the procedures in Reference 10 be followed for inspection of concrete decks.

Concrete beams are usually critical with respect to shear response. The examination of distress in the critical shear areas near beam ends, including the establishment of shear reinforcement size and location, is important. Non-destructive test procedures, such as the use of radiographic equipment or a pachometer may be used to locate reinforcement. The sounding by hammer of all areas of the webs of beams is also extremely important, as initial delamination may indicate critical shear response prior to surface spalls.

Pretensioned and post-tensioned beam-and-slab systems should be surveyed and assessed more critically with respect to cracking. Attention should be paid to secondary effects, and to those areas adjacent to and at post-tensioned anchorage zones. Loss of prestress (i.e., strand breakage), prestress eccentricity, or lack of transverse prestress could cause undesirable tensile forces and longitudinal or transverse cracking in slabs or webs and flanges of beams and boxes. Incorrect placement of steel reinforcement for prestress force containment has been known to cause distress in a number of cases.

Increasing awareness of thermal effects on structures has been noted in current literature [11, 12]. Thermal response may be cumulative with live-load response. Thermal effects are of considerable importance for box systems and for some rigid frames which have unusually stiff substructures.

C14 — 5.3.2 Strength Properties

Concrete components cannot be readily assessed without the use of special equipment and/or destructive examination.

Strength results obtained from ultrasonic pulse velocity or surface hardness methods should not be used by themselves but should be used in conjunction with strength results from concrete core tests.

For a bridge showing signs of deterioration, or where the quality of the concrete is suspect, more than the minimum number of cores may be required to establish the strength of the concrete. Cores should be taken from a component considered for evaluation. When the compressive strength of concrete used in the evaluation is based on core test results, the value chosen should take into account the size of the core.

The specified strength of reinforcing and prestressing steel given in Section 14 — 5.3.2 is based on information obtained from the Reinforcing Steel Institute of Ontario, the Canadian Standards Association, and Canadian steel companies, together with updated information based on past practice in Ontario. Many older types of steel reinforcement have been reworked from rail steels, and show surprisingly high tensile strength when tested.

Wood C14 — 5.4

C14 — 5.4.1 Inspection of Components

Of the many types of deterioration which can affect the strength of wood bridges, decay and rot are the most difficult to assess. Four commonly used methods of testing for decay are: sounding, probing, brace and bit, and increment borer [13]. The reliability of the test methods is directly dependent on the experience of the investigator, as he must anticipate the most likely areas where decay may occur, recognize the type of decay, and know the best method of inspection.

Bridges constructed of pressure-treated material normally suffer only from localized decay. The decay problems are usually limited to members which were improperly treated, severely checked, or which suffer from defects unique to the particular members.

Extra attention should be paid to the possibility of internal decay in bridges built from members which were only submerged in a bath of creosote.

The normal use of sawn timber may result in distress in the form of horizontal cracking on the side(s) of beams, and in longitudinal cracking on the side(s) of compression members.

Metal straps have been added to sawn timber compression members to restrain longitudinal cracking. Both the condition of straps and whether or not the cracks have been properly treated with preservative should be noted.

The inspector should note any timber members in which the grain of the sawn timber does not run parallel to the direction of the member.

A member with a sag should be examined carefully and, if possible, the construction details obtained. A warp may have existed in a member at the time of installation, and the apparent sag or upward camber may have no relationship to loads.

In wood truss bridges, excessive sagging may indicate relaxation of joints. Additional eccentricity may develop, causing a reduction of capacity. Tightening of joints may improve the load-carrying capacity of a truss.

Wood trusses supported along the top of wood cribs are susceptible to differential settlement of the cribs, especially when the back portion of the crib settles more than the front. Such settlement may change the point of support and produce moment in the bottom chord, adjacent to the cribs.

The bottom chord of trusses was frequently notched in the manner shown in Figure C14 — 5.4.1 in order to facilitate connection with the end diagonal member. If no metal reinforcement exists at the end of the notch, close examination is necessary to ensure that forces from the end diagonal will not cause splintering of the bottom chord. Such splintering would result in a failure of the connection between the end diagonal and the bottom chord, or the end diagonal could slip out of the notch.

C14 — 5.4.2 Strength Properties

West Coast Fir, which is considerably stronger than most other species, was used extensively for bridges constructed up to 1950. Its strength as given in Section 13 should be used if identification proves the type to be West Coast Fir.

Bearings **C14 — 5.5**

Bearings normally provide part of the system of structure articulation, and their behaviour should be recognized in the overall evaluation.

A number of different configurations of fixed and expansion-bearing devices are used in bridge construction. Most bearings are fabricated from either steel, aluminum, cast iron, neoprene, teflon (PTFE), rubber, bronze, or a combination of these materials. Inspection procedures and problems associated with bearings and their performance will be given in the maintenance manual being prepared by the Ministry.

Bearing malfunction or distress may influence structure performance; however, it is unlikely that this would lead to posting the structure.

Evaluation Procedures **C14 — 6**

General **C14 — 6.1**

This clause assumes that a condition survey of the bridge has been conducted. When evaluation of a bridge is required for the passage of a controlled vehicle, details of the vehicle and the load being shipped (including axle load, axle spacing and location, tire size and pressure) should be supplied to the owner and engineer responsible for evaluation.

The evaluation should be carried out for dead loads acting simultaneously with live loads. If the structure shows extensive distress due to other transitory loads, such transitory load effects should be included in the overall evaluation. The existing structure restraints and articulation should be carefully defined, as they may play an essential role in the evaluation.

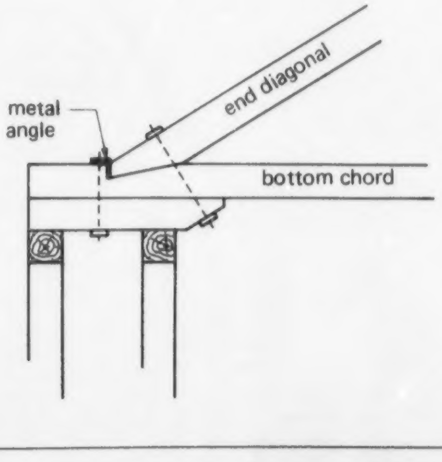
Where it is desirable to improve upon the posting loads, refined analysis methods should be used. Experience indicates that some composite action may be present in concrete deck slab and steel beam structures, even in the absence of mechanical shear connectors, and an allowance for such composite action may be made in the evaluation.

Limit States **C14 — 6.2**

Serviceability limit states need not normally be considered during the evaluation process. The serviceability limit states stated within the design sections of the Code deal with new designs where it is practical to satisfy these conditions. Only serviceability conditions which may lead to the closing of a bridge need be given consideration.

Fatigue in steel bridges may need to be considered. However, it is generally more acceptable to retrofit a bridge for fatigue or cracking than to restrict the loading. Evaluation for load-carrying capacity would be carried out after retrofitting. Aspects concerning welding details, attachment of connections, and their susceptibility to cracking are discussed in References 14 and 15.

Figure **Bottom Chord Notch Detail** **C14 — 5.4.1**



Loading Combinations **C14—6.3**

Most bridge types should be evaluated for loading combination Type 1 only. However, temperature loads may have a significant effect on some continuous rigid frame structures and on some steel structures where the bearings are frozen. Differential temperature gradients may also have a significant effect on large box girders. Since it is not possible to post bridges for temperature or wind loads, the engineer should look into ways of overcoming the effects of these transitory loads through changes in the articulation of the bridge or through repairs.

Sidewalk live loads in addition to vehicular live loads may be included where they are considered to be of importance by the engineer. Guidelines for reducing the posting loads due to the effects of sidewalk live loads are not given but should be based on the relative importance of the loads and the location of the structure.

The reduced live load factor of 1.25 would normally be used when it is desirable to improve the load-carrying capacity of the structure by a few tonnes. This improvement may be sought either to avoid posting of the structure or to increase the posting so that certain types of loads can move over the structure. When the reduced load factor is used, the structure should be inspected at intervals recommended by the engineer and should be re-evaluated within 5 years.

The provisions of Clause 2—6.1.2 are provided as an additional factor of overall safety and may govern in some cases where the lower live load factor of 1.25 is used with lower dead load factors. In these cases, it is deemed desirable to have an overall factor of safety of 1.25 against dead plus live loads. However, this overall factor of safety of 1.25 does not apply to controlled vehicles whose axle spacing and wheel loads are known.

Effects of Deterioration **C14—6.4**

The provisions of this clause relate to the reduction or loss of stiffness of the component, and reduction of capacity due to the partial deterioration of its material.

For some multiple load path structures, an analysis for the redistribution of force effects may be desirable and appropriate.

In some cases of extensive deterioration of the primary components, it may be desirable to evaluate the bridge assuming that those components would be replaced so that the capacity of the rest of the structure may be ascertained.

The section of the component that controls the load-carrying capacity may not be the section that is subject to deterioration, for example:

a) gross sections versus net sections of riveted joints,
b) deterioration of the web at midspan of beams, and
c) deterioration of flanges at simple supports.

Asymmetrical deterioration of a component may induce local eccentricity and instability.

Eccentricities induced by asymmetrical deterioration in tension elements may be neglected. Out-of-alignment of tension members may also be neglected unless it results in abrupt changes of curvature.

When determining the resistance of slender compression components subject to buckling, the standard stability formulae for straight components with uniform cross-section may not be applicable to compression components having severe local deterioration. The engineer should consider the induced local eccentricity and the change of stiffness along the component.

Factored Resistance **C14—6.5**

C14—6.5.1 **General**

For most cases, the factored resistance of existing bridges can be determined in accordance with the sections of the Code which have been developed for new designs. When existing bridge members do not satisfy the design provisions or limitations, the calculation of factored resistance should be carried out using rational methods which are consistent with those used to establish the design section of the Code. An example of this condition is a compression chord with a slenderness ratio greater than 120, or compression elements exceeding the width-thickness limitations outlined in Clause 10—8.2.

C14—6.5.2 **Concrete Deck Slabs**

The graphs in Appendix A14—1 provide a rapid method of evaluating the load-carrying capacity of existing slabs. These graphs are based on the theory outlined in References 16 and 17, and can be used provided that the slab parameters are within the limitations specified in this clause. The span and span/thickness limitations are based on experimental data from model and/or prototype studies. Data are not currently available to extrapolate the theory beyond the span limit of 4.5 m and the span-to-thickness limit of 20. These graphs do not apply to cantilevered slabs.

The slab capacities are based on a wheel load at the centreline of the span of the slab panel. The wheel load considered is a dual wheel, consisting of two 250 mm by 250 mm areas in contact with the deck, separated by a clear space of 100 mm.

The load factors and dynamic load allowance are as given in Section 2. The performance factor for concrete is set at 0.50 and is specified in Clause 14—7.2.2. This more conservative value was selected to ensure an adequate level of safety against a shear failure and to reflect the variations in the actual condition and geometry of existing concrete deck slabs.

C14—6.5.3 **Steel Trusses**

Steel trusses are usually analysed on the assumption that their members are joined through pin connections. In actual trusses, the various members are frequently rigidly connected together so that the end of a member cannot rotate freely. Consideration of the rotational restraint can considerably enhance the load-carrying capacity of the compression members [18]. A more realistic assessment of the capacity of the top chord in pony truss bridges may be obtained by considering the interaction of the top chord with the outstanding verticals and the floor beams to which they are connected [19].

Before considering a more refined analysis of compression members, an analysis of the floor system (i.e., floor beams and stringers) and bottom chord tension members should be conducted. The Ministry's experience in evaluations has shown that these members, in the majority of cases, govern the load-carrying capacity of the bridge.

Solutions for buckling problems assume that a compression member acts as one unit, and that it will not fail through the local buckling of one of its components. In reality, components of a built-up compression member are themselves susceptible to local failure, especially the cover plates which could ripple due to the build-up of rust in the space between the cover plate and the parent section, leading to reduced capacity of the deformed cover plate [20].

C14—6.5.4 **Rivets**

Rivets are no longer used in new construction. However, many riveted steel plate girders and trusses exist on the highway system and must be evaluated. Therefore, provisions for determining the resistance of rivets are included. The value of performance factor chosen for rivets was selected to give the same number of rivets as obtained from service load design methods.

Bridge Posting **C14—7**

General **C14—7.1**

The provisions of this clause relating to single posting based upon engineering judgement and experience should be considered where a meaningful evaluation is not possible. The posting load should be based upon Level 1 loading and should not exceed 18 t. This type of posting for severely deteriorated bridges should be for short time periods of one to two years, which would normally be sufficient time to plan and design the replacement structure.

Concrete bridges that are currently posted and cannot be evaluated due to lack of drawings should be inspected regularly. Reinforced concrete bridges inherently possess higher load-carrying capacities than are predicted by most of the theories generally utilized in bridge engineering. This is the reasoning behind the statement that concrete bridges which show no signs of distress shall not be posted. Also, cracking precedes serious problems with concrete bridges; hence, it is possible to monitor their behaviour under service loading before deciding upon a posting. Tests on reinforced concrete deck slabs of bridges [17, 20] and reinforced concrete rigid frame bridges [22] confirm the presence of sufficient reserve strength in these structures.

Scale-Down Factor, F **C14—7.2**

Scale-down factor F is a factor by which the evaluation live load has to be multiplied so that the factored capacity of the bridge is not exceeded for the combination of dead and live loads under consideration. It is implicit in the equation in Clause 14—7.2.1 that all the wheel loads are multiplied by the same scale-down factor.

C14—7.2.1 **Evaluation Levels 1, 2, 3, and Controlled Vehicles**

For structures whose behaviour is non-linear with respect to loads (for example, columns), the effects of dead loads and live loads cannot be considered separately. For such structures, the smallest value of F will have to be calculated by an iteration process using the equation in Clause 14—7.2.1.

Calculations for Posting Loads C14—7.3

The concept of more than one level of posting loads results from the fact that the variability of traffic configurations cannot be effectively represented by any single posting load as this is generally too restrictive for long vehicles on short-span bridges. A large number of posting loads for a bridge, each corresponding to a different type of vehicle, cannot be effectively transmitted to the driver of a vehicle moving at high speed. For ease of reading the sign, it was decided to limit the number of posting loads to three. The first load would correspond to single unit trucks, the second to tractor-and-one-trailer combinations, and the third to combinations of one tractor and more than one trailer.

A posted bridge should be able to sustain safely any vehicle having a total weight less than, or equal to, the corresponding evaluation load. An OHBD truck multiplied by the scale-down factor F, which satisfies the equation in Clause 14—7.2.1 can indeed be safely sustained by the bridge. However, there could not be a one-to-one relationship between the total weight of the OHBD truck multiplied by the scale-down factor F and the corresponding posting load, since the posting has to account for the following.

a/ **The load shift allowance, L_1** — which allows for the possible differences in wheel loads between the OHBD truck load and actual vehicles, for empty or partly loaded trailers and for uneven distribution of loads in trailers. A posting load has to be such that, within the group, any vehicle having a total load less than, or equal to, the posted load should not cause load effects in any bridge component in excess of those caused by the corresponding OHBD truck load. Load shift allowance, L_1, is given by:

$$L_1 = (F_s - 1.0)\,G_a$$

where: $$F_s = \frac{F_a/G_a}{F_e/G_e}$$

F_a = the maximum force effect due to an actual truck
G_a = the gross weight of the actual truck
F_e = the maximum force effect due to an OHBD truck load
G_e = the gross weight of an OHBD truck.

Table **Posting Loads (Tonnes) Corresponding to Evaluation Level 1 Loads** **C14—7.3(a)**

Scale-Down Factor, F	Equivalent Span, m		
	5	10	15 +
0.30	2	4	5
0.35	3	5	7
0.40	5	7	8
0.45	6	8	10
0.50	8	9	11
0.55	10	11	12
0.60	11	13	14
0.65	13	14	15
0.70	15	16	17
0.75	17	18	18
0.80	19	19	19
0.85	21	21	21
0.90	22	22	22
0.95	24	24	24
1.00	25	25	25

b/ The overload allowance, L_2 — which allows for the possibility of the posting regulation being violated by a small margin. Overload allowance L_2 is given by:

$$L_2 = 35 + \frac{FG_e}{6}\text{, kN, for Level 1}$$

$$L_2 = 30 + \frac{FG_e}{7}\text{, kN, for Level 2}$$

$$L_2 = 30 + \frac{FG_e}{9}\text{, kN, for Level 3}$$

Therefore, the Posting Load, P, is given by:

$$P = FG_e - L_1 - L_2$$

The load shift allowance and the overload allowance are graphically represented in Figure C14 — 7.3 for Evaluation Level 1. Their effects have been allowed for in Figures 14 — 7.3(a), (b) and (c) and in Tables C14 — 7.3(a), (b) and (c).

Posting loads represented in Figures 14 — 7.3(a), (b) and (c) were derived for simple spans, with the load shift and overload allowances based upon the actual axle and gross loads of approximately 9000 vehicles in the 1975 MTC vehicle survey. Single-Trip and Annual Permit vehicles were excluded in the derivation of the figures.

Figures 14 — 7.3(a), (b) and (c) have been converted to Tables C14 — 7.3(a), (b), and (c). These figures and tables are applicable to continuous spans by the use of equivalent span length. This concept is stated in general terms in Clause 14 — 7.3 and is shown in Table C14 — 7.3(d).

In rare cases, if the calculated posting load for Level 2 is less than that for Level 1, then the smaller posting load should be used for both Levels 1 and 2. Similarly, if the calculated posting load is smaller for Level 3 than that for Level 2, then the smaller posting load should be used for Levels 2 and 3.

Table **Posting Loads (Tonnes) Corresponding to Evaluation Level 2 Loads** **C14 — 7.3(b)**

Scale-Down Factor, F	Equivalent Span, m					
	5	10	15	20	30	40 +
0.30	5	5	6	8	9	11
0.35	6	6	7	9	11	13
0.40	8	7	9	11	14	16
0.45	10	9	11	13	16	18
0.50	12	11	13	16	18	20
0.55	15	13	15	18	21	23
0.60	17	16	18	21	23	25
0.65	21	18	21	23	26	27
0.70	24	22	24	26	28	30
0.75	28	25	27	29	31	32
0.80	33	28	30	32	33	35
0.85	37	33	34	35	36	37
0.90	39	37	38	39	39	39
0.95	42	42	42	42	42	42
1.00	44	44	44	44	44	44

Table **Posting Loads (Tonnes) Corresponding to Evaluation Level 3 Loads** **C14 — 7.3(c)**

Scale-Down Factor, F	Equivalent Span, m							
	5	10	15	20	30	40	50	60 +
0.30	16	16	16	16	16	16	16	16
0.35	18	18	18	18	18	18	18	19
0.40	20	20	20	20	20	20	21	22
0.45	22	22	22	22	22	23	24	25
0.50	23	23	23	23	24	25	27	29
0.55	25	25	26	26	27	28	30	32
0.60	26	27	28	28	29	32	33	35
0.65	27	28	30	31	32	34	37	38
0.70	28	30	32	33	35	37	39	41
0.75	30	33	35	36	38	40	42	44
0.80	38	39	40	40	41	43	45	47
0.85	47	51	51	44	45	46	47	51
0.90	54	54	54	50	50	50	52	54
0.95	57	57	57	57	57	57	57	57
1.00	60	60	60	60	60	60	60	60

Figure **Post Loads Corresponding to Evaluation Level 1 Loads** **C14 — 7.3**

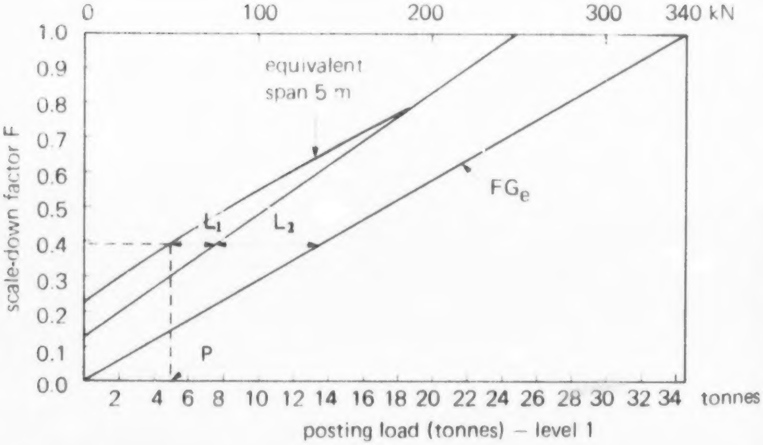
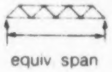
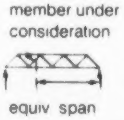
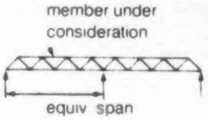
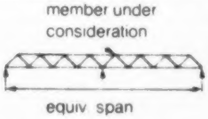
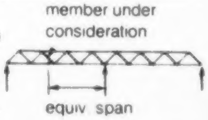
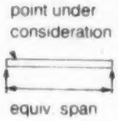
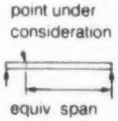
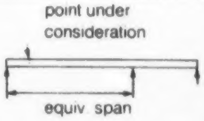
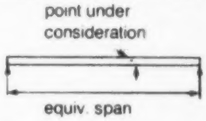
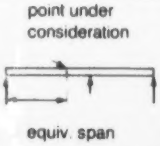


Table **Equivalent Spans** **C14 — 7.3(d)**

Type of Bridge	Number of Spans	Response under Consideration	Equivalent Spans to be used in Conjunction with Figure A14 — 1(a), (b) and (c)	
Truss and Arch Bridges	One	Load effect in a top or bottom chord member	Span Length	
		Load effect in a unit member	The larger of the distances between the centre of the member to the supports.	 member under consideration equiv. span
	Multi	In a top or bottom chord member in the positive-moment region of the span	Span Length	 member under consideration equiv. span
		In a top or bottom chord member in the negative-moment region of the spans	Sum of the two adjacent spans	 member under consideration equiv. span
		Load effect in a unit member	The larger of the distances between the centre of the member to the two adjacent supports.	 member under consideration equiv. span
All Bridges Other Than Truss and Arch Bridges	One	Moments	Span Length	 point under consideration equiv. span
		Shears	The larger of the distances between the point under consideration and the supports	 point under consideration equiv. span
	Multi	Moments in positive moment regions.	Span Length	 point under consideration equiv. span
		Moments in negative moment regions.	Sum of the two adjacent spans.	 point under consideration equiv. span
		Shears	The larger of the distances between the point under consideration and the two adjacent supports.	 point under consideration equiv. span

C14 — 7.4.1 **Calculations for Controlled Vehicle**

If the scale-down factor F is less than 1 for a controlled vehicle then the bridge is not capable of carrying that vehicle. In such cases it may be desirable to strengthen the bridge.

Posting Signs **C14 — 7.5**

C14 — 7.5.1 **Single Posting Sign**

Details of the posting sign should be in accordance with the Ministry's *Sign Manual* [23]. A single posting is shown in Figure C14 — 7.5.1

C14 — 7.5.2 **Triple Posting Sign**

Details of the triple posting sign shall be in accordance with the Ministry's *Sign Manual* [23]. A triple posting sign is shown in Figure C14 — 7.5.2.

Load Testing **C14 — 8**

Load testing is an effective means of evaluating the structural performance of a bridge or its components. This applies particularly to bridges which cannot be accurately modelled by analysis, or if the structural response of a bridge to live load is in question. A load test should only be carried out if the authority believes that it would provide a more realistic and safe load-carrying capacity of the bridge. A condition survey and an analysis identifying critical components in the bridge should be carried out prior to any load test. During a load test, critical components should be monitored to ensure that additional deterioration or distress does not develop as a consequence of each load increment up to the proposed maximum level. Examples of load tests of bridges are given in References 24 to 29.

Figure **An Example of the Single Posting Sign** **C14 — 7.5.1**

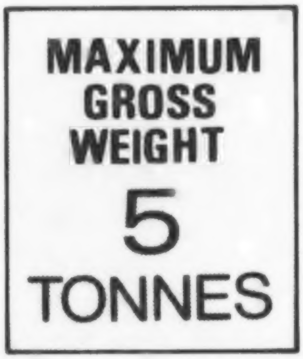
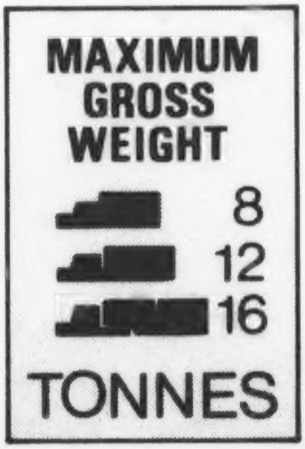


Figure **An Example of the Triple Posting Sign** **C14 — 7.5.2**



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